STAT

HANDBOOK OF HYDRAULICS

STAT

MILITARY HYDROLOGY BULLETIN 12
JUNE 1957

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BY

MILITARY HYDROLOGY R & D BRANCH
U. S. ARMY ENGINEER DISTRICT, WASHINGTON

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2. Hydrology
3. Military Operations

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PREFACE

This bulletin is one of a series dealing with the various aspects of hydrology and hydraulics involved in military operations and with the techniques and methods of analysis which are considered most suitable for army use. A number of these techniques were developed in the course of Research and Development Project No. 8-97-10-003, assigned to the U. S. Army Engineer District, Washington on 14 March 1951 by the Chief of Engineers. Printing of this bulletin was authorized by the Office, Chief of Engineers, on 9 May 1957.

Mr. A. L. Cochran of the Office, Chief of Engineers, formulated the objectives and scope of this bulletin. Messrs. W. B. Craig and J. T. Gay of the Military Hydrology R&D Branch, U. S. Army Engineer District, Washington, prepared the bulletin under the supervision of Mr. R. L. Irwin.

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SUMMARY

Data necessary for the solution of hydraulic problems encountered in military hydrology are scattered throughout engineering literature. While most engineering manuals and texts are concerned with features of design, the military hydrologist is primarily concerned with the functional analysis of existing structures. This handbook, therefore, attempts to assemble hydraulic information useful to the military hydrologist, and to explain the methods used in solving some of the hydraulic problems of military hydrology.

The material presented in this bulletin has been selected from hydraulic data found in the publications of various technical societies, engineering textbooks, and from the results of hydraulic research and field observations.

The equations and methods presented are intended to give results within the degree of accuracy required for military hydrology purposes. It is fully recognized that equally satisfactory and accurate results may be obtained by the use of formulas and data other than those presented. The methods of computation described herein may be shortened if expediency warrants by substituting experience and engineering judgment for the detailed computation procedures, thereby reducing the time of computation. The bulletin does not cover the entire subject of hydraulics and will not make a qualified hydraulic engineer out of the individual soldier.

This bulletin is of a tentative nature, designed to make available to the armed forces such useful information as could be compiled in a prescribed period of time. It is believed the bulletin offers to military agencies useful information in military hydrology that can be incorporated in standard operating procedures. The use of this bulletin under field conditions will point up deficiencies and provide a basis for determining the requirements for a final manual.

CHAPTER I
INTRODUCTION

SECTION A: INTRODUCTORY STATEMENTS

1. Purpose. The purpose of this bulletin is to present in convenient form a handbook of hydraulics, useful in the solution of problems pertinent to military hydrology.

2. Scope. This bulletin contains a brief review of the fundamental principles of hydraulics and the application of these principles to the analysis of specific hydraulic structures which are encountered in military planning and field operations of modern warfare. The types of structures considered include canals, levees, culverts, and high or low head dams with gated or ungated spillways and outlets. Useful data are assembled in tabular and graphic form to reduce the labor of computation and expedite the work in the field.

3. Arrangement. a. The bulletin is generally divided into two parts. Chapters I through V present the fundamental principles of hydraulics. Chapters VI through IX outline the methods used in determining the discharge rating curves for specific hydraulic structures. The arrangement is as follows:

Chapter I	Introduction
Chapter II	Orifices, Tubes, and Culverts
Chapter III	Weirs
Chapter IV	Pipe Flow
Chapter V	Open Channels
Chapter VI	Spillways
Chapter VII	Navigation Dams
Chapter VIII	Reservoir Outlet Conduits
Chapter IX	Headwater Control

b. Paragraph numbering is consecutive throughout the entire bulletin.

c. Plates are numbered with a three-digit system; the first number corresponds to the chapter number, the last two represent the plate sequence number within the chapter. Where a plate consists of several sheets, these are designated by capital letters following the plate number. For example, Plate 205A is the first sheet of the fifth plate of Chapter II. The plates for each chapter are assembled at the end of the chapter concerned.

d. All references in each chapter are assembled at the end of the chapter concerned. The references are numbered consecutively in each chapter starting with reference number /1/. The reference numbers appear in the text and refer to the reference numbers at the end of each chapter. Associated Military Hydrology publications are listed in Appendix C at the end of this bulletin.

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SECTION B: HYDROSTATICS

4. Application of Hydrostatics. The calculation of the magnitude, direction, and location of the total forces on a submerged surface, is essential to solving many problems encountered in military hydrology.

5. Pressure Variation in Steady Uniform Flow. a. Under conditions of steady, uniform flow a liquid does not accelerate in any direction and the sum of p + Z equals a constant:

$$p + \gamma Z = C \qquad (1\text{-}1)$$

where:

C = a constant of integration
γ = specific weight
Z = elevation of a fluid element above a datum plane
p = pressure intensity of the fluid at elevation Z

b. This indicates that the sum of the pressure and γZ will be the same at all points within a fluid which undergoes no acceleration. Since the pressure can vary only if γZ varies, it follows that under conditions of zero acceleration, the pressure intensity will have the same magnitude at all points of equal elevation in the same fluid. Also the pressure will increase from point to point in a fluid by an amount equal to the corresponding decrease in γZ or vice versa.

6. Piezometric Head. a. Equation 1-1 neglected the tangential stresses due to fluid deformation. Therefore, the equation is correct for either hydrostatic conditions or conditions in which the velocity is equal at all points in the fluid with zero fluid deformation. Under many conditions of fluid motion, the effects of acceleration and tangential stresses are negligible in comparison with other factors. For such conditions, the assumption of hydrostatic pressure distribution is adequate to determine the force of a fluid on the boundaries. Under conditions of liquid flow with hydrostatic pressure distribution the piezometric head is defined as the sum of $p/\gamma + Z$ as follows:

$$h = \frac{p}{\gamma} + Z \qquad (1\text{-}2)$$

b. In flow with a free surface and hydrostatic pressure distribution, the pressure head at any point is equal to its depth below the surface. In a closed conduit the piezometric head "h" represents the height to which a liquid would rise in an open tube connected to a piezometer orifice in the boundary.

7. Hydrostatic Pressure on Submerged Plane Surfaces. a. The total force due to liquid pressure upon a plane surface is equal to the product of the specific weight of the liquid, the area of the surface, and the depth of its centroid below the free surface or piezometric plane

(Fig. 1, Plate 101). This relationship is determined by the following equation:

$$F = \gamma \, \bar{h}_p A \qquad (1\text{-}3)$$

where:

F = total force in pounds
γ = specific weight (for water, 62.5 lb/cu. ft.)
$\bar{h}_p$ = head, from the water surface to the centroid of the submerged area in ft.
A = area in sq. ft.

b. The distance between the line of intersection of a plane boundary surface with the free surface of the liquid and the line of action of the resultant hydrostatic force (the distance "y_{cp}" to the center of pressure from x-x shown in Fig. 1, Plate 101) is equal to the ratio of the second and first moments of the area about the line of intersection:

$$y_{cp} = \frac{I_x}{O_x} = \bar{y} + \frac{\bar{I}}{\bar{y}A} = \bar{y} + \frac{k^2}{\bar{y}} \qquad (1\text{-}4)$$

where:

y_{cp} = distance to the center of pressure from the water surface along the prolongation line of the submerged surface
$\bar{y}$ = distance to centroid from water surface along the prolongation line of the submerged surface
k = radius of gyration about a horizontal axis through the centroid of the submerged surface
$\bar{I}$ = moment of inertia about the centroidal axis parallel to the water surface
I_x = moment of inertia of the submerged surface about the x-x axis
O_x = moment of area or first moment of the submerged area about the x-x axis.

c. Fig. 2, Plate 101, shows the more common shapes encountered in hydraulic problems, with the vertical distance z from the base to the center of gravity (cg).

8. Hydrostatic Pressure on Submerged Curved Surfaces. a. The horizontal component of the total hydrostatic force upon a curved surface, is equal to the force which would be exerted on the vertical projection of the surface (as shown by line EA in Fig. 3, Plate 101) and has the same line of action as the force on the vertical projection.

b. The vertical component of the total hydrostatic force upon a curved surface (Fig. 3, Plate 101) is equal to the weight of a liquid column extending from the boundary surface to the free surface (ABCD). The line of action passes through the center of gravity of this liquid column.

SECTION C: FLUID FLOW

9. Application of Fluid Flow. Hydraulic problems encountered in military hydrology are concerned with water flowing with turbulent motion (i.e., motion in which the direction and magnitude of the velocity vary irregularly with time). Most of the problems may be solved by considering the flow as characterized at each section by a mean velocity, mean pressure, and a mean elevation.

10. Definitions. a. Discharge. The volume of fluid passing a cross section of a channel in unit time is called the discharge. The symbol Q is used to designate the discharge, the usual units being cubic feet per second (cfs). If V is the mean velocity in feet per second past any cross section, and A is the cross sectional area in square feet

$$Q = AV \qquad (1\text{-}5)$$

b. Steady and Unsteady Flow. Fluid flow may be steady or unsteady. Steady flow occurs in a system when none of the variables involved changes with time. If any variable changes with time, the condition of unsteady flow exists. If the discharge Q passing a given cross section of a stream is constant with time, the flow is steady at that cross section. Examples of unsteady flow are discharges through orifices and over spillways under falling heads, or the passage of a flood wave down a channel.

c. Uniform and Non-Uniform Flow. The flow is said to be uniform in a reach if, with steady flow in any length, the average velocity at every cross section is the same. If the velocities are not the same, the flow is non-uniform. Uniform flow is possible only in a channel of constant cross section. Non-uniform flow occurs in all channels where there is either accelerated flow or backwater.

11. Equation of Continuity. In steady flow, uniform or non-uniform, the quantity per second, Q, passing any section must be constant. If A_1 and V_1 represent the cross-sectional area and average velocity at a section 1, and A_2 and V_2 represent the same quantities at a section 2, then

$$Q = A_1V_1 = A_2V_2 \qquad (1\text{-}6)$$

thus, the mean velocities at all cross sections having equal areas are then equal. If the areas are not equal, the velocities are inversely proportional to the areas of the respective cross sections.

12. Energy. a. Kinetic and Potential Energy. The total energy possessed by a fluid consists of the kinetic energy and the potential energy of the fluid. The kinetic energy of the fluid of mass m and moving with velocity V, is $m\,V^2/2$. The potential energy of the fluid results from the position of the fluid element and the pressure on

the fluid element, and is equal to $Z + p/\gamma$. The total energy of a fluid element per unit weight is expressed by the equation

$$E = \frac{p}{\gamma} + Z + \frac{V^2}{2g} \qquad (1\text{-}7)$$

where: E = the total energy per pound of fluid in the fluid element
p = pressure, pounds per square ft.
= specific weight of the fluid, pounds per cubic foot
Z = elevation of the fluid element above a given datum in feet
V = velocity of the fluid element in feet per second
g = acceleration of gravity in ft. per sec^2.

b. Specific Energy. The energy in a cross section of an open channel with the datum reference plane coincident with the channel bottom, is called the specific energy. The specific energy of an open channel is computed by the following equation:

$$H_o = y + \frac{V^2}{2g} = y + \frac{Q^2}{2gA^2} \qquad (1\text{-}3)$$

The specific energy, H_o, of a stream thus equals the depth of water plus the velocity head.

c. A certain discharge may flow in an open channel in a number of ways, each characterized by a different depth. To each depth there corresponds a definite velocity head for the given discharge and therefore a definite value of the specific energy. A specific energy diagram may be drawn plotting the depth of flow in a given cross section against the corresponding specific energy for the given discharge. A typical specific energy diagram is shown on Fig. 1, Plate 102. The potential energy is represented by the line 0-a which is 45° to the x axis. The velocity head or kinetic energy head curve is asymptotic to the x and y axis as shown by curve b-c. The sum of the curves is the specific energy curve d-e. It will be noted that the specific energy curve has two depths for each value of the specific energy, except at the point of minimum energy. This depth of flow at which the specific energy is a minimum is termed the critical depth and will be discussed more fully in Chapter V.

d. A distinction should be noted between the energy defined by equation 1-7 and the specific energy as defined by equation 1-8. The energy in equation 1-7 is energy referred to a constant datum line and indicates the changes in energy over a certain length of flow as a whole. The specific energy in equation 1-8 is the energy referred to the channel bottom which changes from section to section. Steady uniform flow is characterized by the specific energy being a constant along the channel.

e. The Energy Equation. (1) Bernoulli deduced that in any stream or conduit flowing steadily without friction, the total energy contained in a given mass is the same at every point in its path of flow. Bernoulli's theorem would be expressed by the following equation:

$$E = \frac{p_1}{\gamma} + Z_1 + \frac{V_1^{\,2}}{2g} = \frac{p_2}{\gamma} + Z_2 + \frac{V_2^{\,2}}{2g} + h_f \qquad (1\text{-}9)$$

where H = the total energy head
h_f = the friction head loss
P_1/γ & P_2/γ = the pressure head at sections 1 and 2
Z_1 & Z_2 = the elevation head above a given datum at sections 1 and 2
$V_1^2/2g$ & $V_2^2/2g$ = the velocity head at sections 1 and 2.

(2) Equation 1-9 is expressed in terms normally used in pipe flow computations. The flow in open channels, when the cross section is either uniform or changes gradually and when the velocities are uniformly distributed, would be expressed by the following equation:

$$H = y_1 + Z_1 + \frac{V_1^2}{2g} = y_2 + Z_2 + \frac{V_2^2}{2g} + h_f \qquad (1\text{-}10)$$

(3) Fluids in motion invariably suffer a loss of energy through friction. In order to make equations 1-9 and 1-10 balance, the frictional head loss between sections 1 and 2 was added to the right-hand side of the equations. If energy is added to the flow between sections 1 and 2, such as by a pump, the equivalent energy head must be added to the energy of section 1. Conversely, if energy is given up by the flow between sections 1 and 2, such as to a turbine, the equivalent energy head must be added to the right-hand side of the energy equation. This would be expressed as follows:

Energy in fluid at Section 1	+	Energy added to fluid between Sections 1 and 2	-	Energy taken from the fluid between Sections 1 and 2, including friction loss	=	Energy in fluid at Section 2

13. <u>Momentum</u>. a. The momentum of a body is the product of its mass and velocity. Impulse is the product of the resultant force acting on a body and the time of action of the force. Expressed as a formula:

$$\text{Momentum} = m\,V$$
$$\text{Impulse} = F\,t$$

b. The force (F) acting upon any mass (m) produces an acceleration (a) or F = ma. If (F) is constant and the average rate of acceleration = $(V_2 - V_1)/t$ where t = time in seconds, then:

$$F = \frac{m(V_2 - V_1)}{t}$$

$$Ft = m(V_2 - V_1) \qquad (1\text{-}11)$$

This may be expressed as follows - "During any period of time, the impulse of the resultant force acting upon a body is equal to its change of momentum."

c. In Fig. 2, Plate 102, the sum of the forces (F) acting is given by the expression:

$$F = F_1 - F_2 = \frac{Q\gamma}{g}(V_2 - V_1) \qquad (1\text{-}12)$$

where: F_1 = the total hydrostatic force at 1-1 = $A_1\bar{y}_1$

F_2 = the total hydrostatic force at 2-2 = $A_2\bar{y}_2$

Equation 1-12 may be expressed as follows - "The change in momentum of a fluid body is equal to the total unbalanced force acting on the body." This will be more fully discussed in Chapter V.

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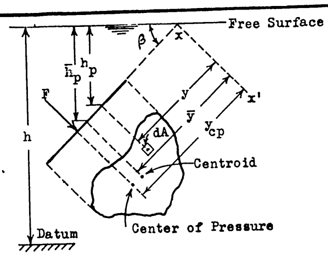

Fig. 1

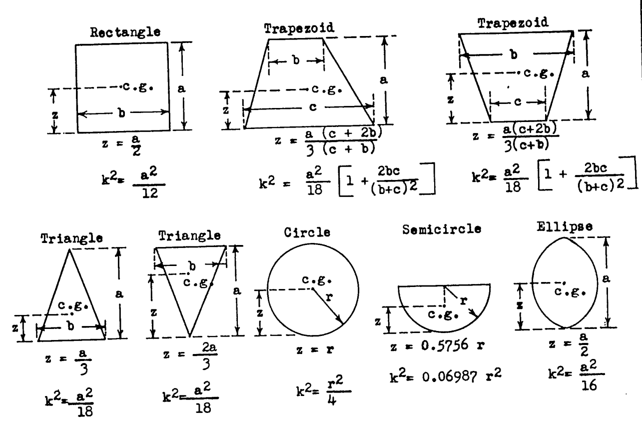

Fig. 2

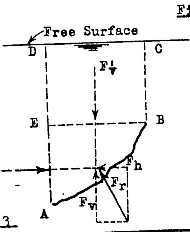

Fig. 3

HYDROSTATICS

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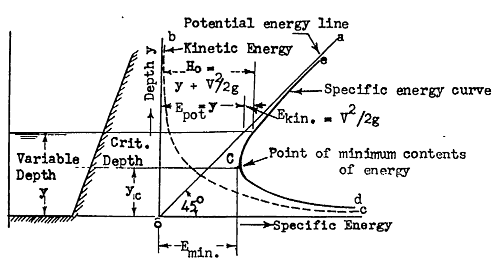

Fig. 1
THE SPECIFIC ENERGY DIAGRAM $H_o = y + \frac{Q^2}{2gA^2}$

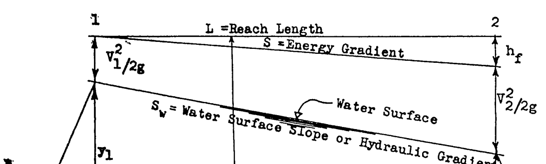

Fig. 2
CONSERVATION OF ENERGY AND MOMENTUM
AND
OPEN CHANNEL GEOMETRY DIAGRAM

ENERGY DIAGRAMS

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CHAPTER II
ORIFICES, TUBES, AND CULVERTS

SECTION A: ORIFICES AND TUBES

14. Basic Equations. a. An orifice is a regular-shaped opening with a closed perimeter through which water flows. If the perimeter is not closed, or if the opening flows only partially full, the orifice becomes a weir.

b. An orifice with prolonged sides, such as a short piece of pipe set in the side of a reservoir is called a tube. An orifice in a thick wall has the hydraulic properties of a tube. The actual discharge of a stream or jet issuing from an orifice is different from the theoretical discharge because of friction loss and colntraction or flaring effects. The discharge from an orifice is given by the formula:

$$Q = C_q A(2gh)^{0.5} \qquad (2\text{-}1)$$

where:
- Q = discharge in cubic feet per second
- A = area of orifice opening in square feet
- C_q = the discharge coefficient
- h = head, in feet, on center line of orifice with free discharge or difference in upstream and downstream water levels for a submerged orifice
- g = the acceleration of gravity

When the velocity of approach is appreciable, the following formula should be used for the computation of the discharge:

$$Q = C_q A \left[2g(h + h_a)\right]^{0.5} \qquad (2\text{-}2)$$

where h_a = the approach velocity head, in feet.

c. Orifices Under Low Heads. If the head on the upper edge of a rectangular orifice is equal to or less than the height of opening, the orifice is considered to be under a low head. The following formula gives the discharge for a rectangular orifice under low heads:

$$Q = 2/3 C_q L(2g)^{0.5}(h_2^{1.5} - h_1^{1.5}) \qquad (2\text{-}3)$$

where:
- L = width of orifice
- h_1 = the head from water surface to top of orifice
- h_2 = the head from water surface to bottom of orifice
- C_q = a coefficient of discharge taken from Plate 202

15. Discharge Coefficient. The coefficient of discharge of an orifice is a function of the boundary gometry, and the ratio of the head on the orifice to its size. The general shape and sharpness of the upstream edge of the orifice will also affect the value of the coefficient of discharge. The variation of the discharge coefficient of a sharp-edged slot or orifice /1/, as a function of the boundary geometry, is shown on Plate 201. The discharge coefficients, /3/, /4/, of various types of orifices and tubes are shown on Plates 202 and 203.

16. Nozzles. A nozzle is a converging tube attached to the end of a pipe or hose which serves to increase the velocity of the issuing jet. At high velocities, C_q(Eq. 2-1) is usually within 2 or 3 percent of unity. The following mean value of coefficients of discharge for smooth nozzles, having a diameter at the base of 1.55 in., were determined from experiments /2/:

Ratio $\frac{\text{Nozzle tip diameter}}{\text{Nozzle base diameter}}$ =	0.485	0.565	0.645	0.725	0.806	0.886
C_q =	0.983	0.982	0.980	0.976	0.971	0.959

SECTION B: CULVERTS

17. Types. a. A culvert /5/ can operate under five sets of conditions as designated by the following types of flow:

- Type I. Part full with free outfall
- Type II. Part full with outfall partially submerged
- Type III. Full with outfall completely submerged
- Type IV. Full with outfall partially submerged
- Type V. Full with free outfall

Of these five conditions, the first three are stable and of primary practical importance. The last two are stable within limits which have not yet been well defined.

b. Free Flow. Type I flow occurs when the culvert flows part full with a tailwater elevation below the invert of the culvert at the outlet. This condition of flow is represented by point 1 on the upper discharge diagram of Plate 204. The coordinates of the discharge diagrams are expressed as ratios of the headwater and tailwater depths, to the diameter of the culvert. The relation between pond elevation and discharge is well defined and stable for Type I flow conditions. The culvert slope for this type of flow, however, must exceed the slope required to overcome friction losses induced by the roughness of the culvert walls. If the rate of discharge remains constant, and if the tailwater level is raised, the headwater remains unchanged until the tailwater level approaches the level of the headwater pond, or reaches the top of the pipe at the outlet. This is represented by the line 1-2 on Plate 204 and is classified as Type II flow.

c. Submerged Flow. As the tailwater level is further raised to submerge the pipe outlet and fill the conduit, the flow is classified as Type III flow and is represented by the line 3-4 on Plate 204. The rate of discharge is then a function of "h_d" which is the difference in pond levels between headwater and tailwater pools. Within the range 3-4 any change in the elevation of tailwater level is promptly reflected in an equal change of headwater level assuming the rate of discharge to remain constant. As the pipe changes from part-full flow to full flow, as represented by the line 2-3, the headwater depth may drop abruptly as shown on Plate 204, or it may rise gradually. The transition 2-3 occurs when the tailwater level is at or near the crown of the culvert at outlet.

d. Unstable Flow Conditions. Type IV flow conditions occur if the culvert has been flowing full, and if the tailwater level is lowered below the crown of the culvert at the outlet. The headwater pool level is also lowered -- but at a rate which decreases proportionately as the tailwater is lowered. This is represented by the range 3-5 on Plate 204, and it represents a zone of operation which may be quite unstable. Point 5 represents the limiting condition in which the outfall is discharging freely with the pipe flowing full throughout its length, and is classified as Type V flow which is also quite unstable.

18. Discharge Capacity. a. The discharge capacity of a culvert is in a category between a weir or short tube as one limit, and a long pipe as the other limit. The discharge capacity of a culvert is a function of at least seven variables. These variables are as follows:

(1) The material of the culvert which effect is reflected by the roughness coefficient
(2) Diameter or cross section of the culvert
(3) Slope of the conduit invert
(4) Entrance conditions as reflected in the entrance loss coefficient for the headwall shape
(5) Headwater elevation above the invert
(6) Tailwater elevation above the invert at the outlet
(7) Outlet conditions as reflected in the means to convert some of the kinetic energy into potential energy below the culvert

b. Free Flow. The discharge capacity of a culvert for free flow conditions, with the hydraulic control at the entrance, is a function of the critical depth. This condition requires that the culvert be set on a slope greater than the normal slope for the maximum discharge, and that the tailwater does not submerge the outlet. The discharge capacity of a circular pipe with square-edged entrance and free-flow conditions would be a function of the diameter to the five-halves power. Plate 205 shows the relationship of the ratio of y_1/D and the discharge factor $Q/D^{2.5}$. This plate would be used to determine the discharge under free flow conditions from a circular pipe, knowing the diameter and head on the culvert. Two nomographs /5/, /6/, are shown on Plates 206 and 207 for use in determining the discharge through a circular culvert or a rectangular culvert. The nomographs are used in determining the discharge for culverts with a square-edged entrance, and the control section at the inlet. A straight line drawn between the variables shown on any two scales would intersect the values of the variables on the other scales.

c. Submerged Flow. The discharge capacity of a culvert, for submerged flow conditions, is a function of the entrance loss, the velocity head loss, and the friction head loss through the conduit. The total head loss through a submerged culvert would be computed by the following equations:

Circular Culvert: $$h_d = (1 + K_e + \frac{fL}{D})\frac{V^2}{2g} \qquad (2\text{-}4)$$

Box Culvert: $$h_d = (1 + K_e + \frac{fL}{4R})\frac{V^2}{2g} \qquad (2\text{-}5)$$

where h_d = head loss in feet between the tailwater and headwater
D = diameter of circular culvert
K_e = entrance loss coefficient
= 0.50 for square edged entrance, $r/D = 0$
= 0.25 for rounded entrance, $r/D = 0.05$
= 0.10 for rounded entrance, $r/D = 0.2$
r = radius of rounding of entrance of conduit in ft.
L = length of culvert in ft.
f = friction factor
= $190\ n^2/D^{1/3}$ for circular culverts
= $116\ n^2/R^{1/3}$ for box culverts
R = hydraulic radius of box culvert = A/P
A = cross-sectional area of culvert
P = wetted perimeter of culvert
$V^2/2g$ = velocity head with culvert flowing full
n = Manning's roughness coefficient

d. Culvert Flowing Full, Outlet Unsubmerged. The discharge capacity of a culvert, for unstable conditions with the culvert flowing full and with the outlet unsubmerged, is a function of the total head loss, the elevation of the hydraulic gradient at the outlet, and the bottom slope of the culvert invert. The total entrance head required to convey a given discharge through the culvert would be equal to:

Culvert Sloping in Direction of Flow

$$y_1 = h_d + Z - a \qquad (2\text{-}6)$$

Culvert Sloping Adversely to Direction of Flow

$$y_1 = h_d + Z + a \qquad (2\text{-}7)$$

where y_1 = the upstream water surface depth above the culvert invert in feet
h_d = the total head loss in feet for a culvert flowing full, equal to the head loss for a submerged culvert as given in Eq. 2-4 or Eq. 2-5
Z = the elevation of the hydraulic gradient above the culvert invert at the outlet and equal to 0.6 D in feet
a = the total fall in the culvert invert in feet.

19. <u>Discharge Rating Curve</u>. a. The discharge rating curve of a culvert structure sloping in the direction of flow would be computed as follows:

(1) Determine the tailwater rating curve below the culvert by methods given in Par. 86.

(2) Determine the normal discharge of the culvert flowing full with the friction slope equal to the bottom slope by the methods described in Par. 71. The discharge is the maximum flow that would normally be released with the control at the inlet. Discharges greater than the normal discharge would cause the culvert to flow full and would be computed by equation 2-4 or equation 2-5.

(3) Assume a discharge through the culvert and determine the tailwater elevation from the tailwater rating curve.

(4) Compute the head loss through the culvert by Eq. 2-4 or 2-5 if the tailwater submerges the culvert outlet.

(5) Compute the head for the assumed discharge by means of the nomograph on Plate 205 if the tailwater does not submerge the outlet.

(6) Assume other values of discharge and repeat steps (3) through (5). Plot the values of the head and discharges which form the points of the discharge rating curves.

b. The flow characteristics of a culvert structure sloping adversely to the direction of flow are difficult to determine. The head necessary to pass a given discharge through the culvert is a function of the head loss due to friction, the elevation of the hydraulic gradient at the outlet, and the entrance losses.

c. When a culvert outlet is not submerged, the pipe may or may not flow full depending on the total head loss through the culvert. For low discharges the culvert would flow as an open channel. The upstream depth for low discharges would be equal to the sum of the initial depth, the drop in the invert of the culvert, and the fall in the water surface due to friction in the culvert, and the entrance loss due to form losses. For moderate discharges the culvert would flow full at the inlet and partially full at the outlet. The upstream depth would be equal to the sum of the critical depth, the drop in the culvert invert, the drop in the hydraulic gradient, and the entrance loss.

d. For practical purposes in military hydrology, assume the culvert flows full for all discharges when the slope of the culvert is adverse to the direction of flow.

e. The discharge rating curve of a culvert structure sloping adversely to the direction of flow would be computed as follows:

(1) Determine the tailwater rating curve for the culvert.
(2) Assume a discharge through the culvert and determine the tailwater elevation from the tailwater rating curve.
(3) Compute the head loss through the culvert by equation 2-4 or 2-5 if the tailwater submerges the culvert outlet.
(4) Compute the head loss through the culvert by equation 2-7 if the outlet is not submerged.
(5) Assume the values of discharge and repeat steps (2) through (4). Plot the values of the head and discharges and draw the discharge rating curve through the points.

20. <u>Examples</u>. The computation of the discharge rating curve of a 48" circular concrete culvert 50 feet long, sloping in the direction of flow, is given on Plate 208, as an example. A second example is given on Plate 209 with the culvert sloping adversely to the direction of flow.

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/5/ Mavis, Fredrick T. - "The Hydraulics of Culverts". The Pennsylvania State College Bulletin, 56. 1943

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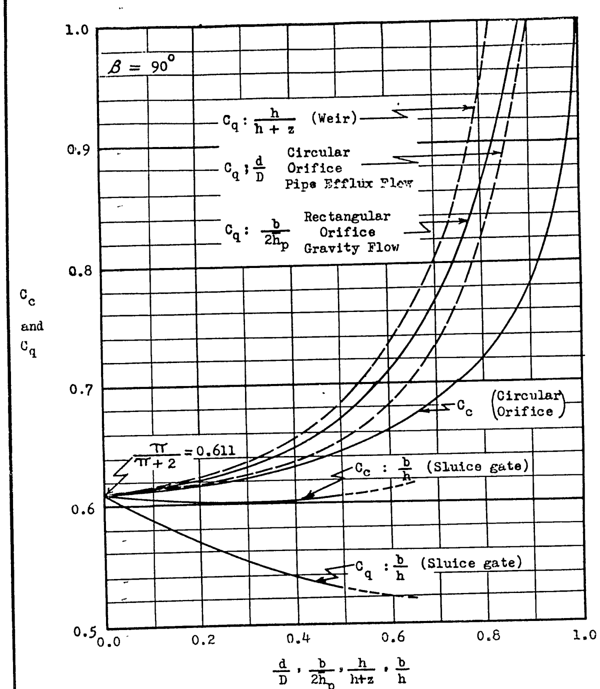

d = Diameter of orifice
D = Diameter of pipe
b = Height of gate opening
h = Head above the gate seat or head on weir.
$\bar{h}_p$ = Depth from water surface to centroid of area or opening.
z = Weir crest height.

DISCHARGE & CONTRACTION COEFFICIENTS

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DISCHARGE COEFFICIENTS FOR VARIOUS TYPES OF ORIFICES AND TUBES

SECTION VIEW	DESCRIPTION	AVERAGE DISCHARGE COEFFICIENT C_q
a	SHARP-EDGED ORIFICE The stream is contracted to about 0.62 of the area of the opening.	0.61
b	ORIFICE OR SHORT TUBE WITH WELL-ROUNDED ENTRY (BELL-MOUTH) The stream is about the same size as the opening.	0.98
c	SHORT TUBE OR ORIFICE WITH SQUARE EDGED ENTRY. When jet springs clear of the walls the flow is the same as for a sharp-edged orifice	0.61
d	SHORT TUBE WITH ROUNDED ENTRY	0.90
e	RE-ENTRANT TUBE When length is about one diameter it is called Borda's mouthpiece. Jet clears walls of the tube	0.52
f	RE-ENTRANT TUBE Length about 2.5 diameters. Flowing full.	0.72 to 0.80

SECTION VIEW	DESCRIPTION	AVERAGE DISCHARGE COEFFICIENT C_q	
g	SHORT TUBE OR ORIFICE WITH SQUARE EDGED ENTRY. Flowing Full. When length of tube is 2.5 diameters it is called a standard short tube	L/D	C_q
		0	0.60
		0.25	0.63
		0.50	0.67
		0.75	0.72
		1.00	0.76
		1.50	0.79
		2.50	0.80
		3.50	0.80
h	INCLINED SHORT TUBE WITH SHARP CORNERED ENTRANCE	θ	C_q
		90°	0.82
		80°	0.80
		70°	0.78
		60°	0.76
		50°	0.75
		40°	0.73
		30°	0.72
i	CONVERGENT SHORT TUBE Sharp Corner At Entrance	ø	C_q
		0°	0.82
		5.75°	0.94
		11.25°	0.92
		22.50°	0.85
j ROUND-CORNERED ENTRANCE		ø	C_q
		0°	0.97
		5.75°	0.95
		11.25°	0.92
		22.50°	0.88
		45.00°	0.75

DISCHARGE COEFFICIENT ORIFICES & TUBES

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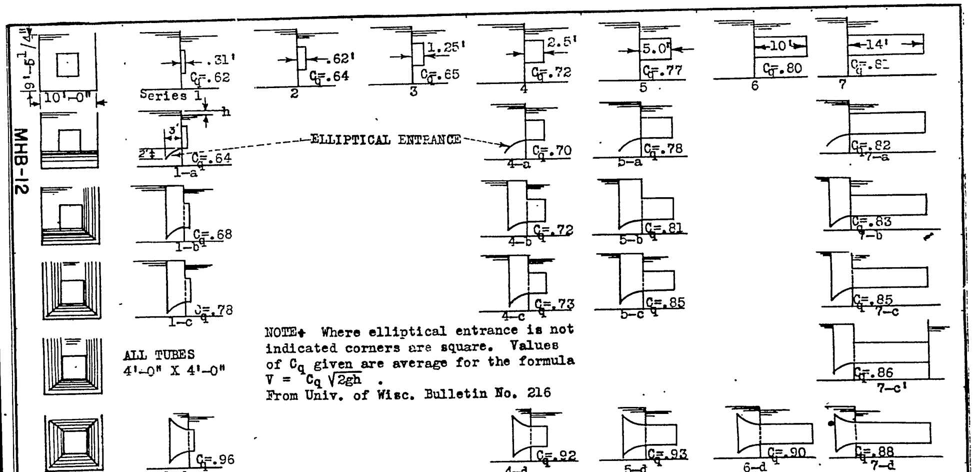
PLATE 202

MHB-12



ALL TUBES 4'-0" X 4'-0"

NOTE: Where elliptical entrance is not indicated corners are square. Values of C_q given are average for the formula $V = C_q\sqrt{2gh}$.
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SUBMERGED TUBE DISCHARGE COEFFICIENT

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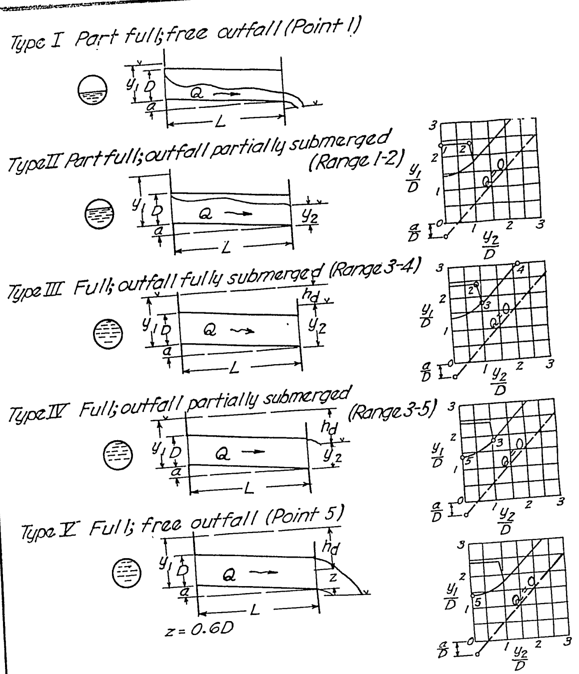
PLATE 203



CULVERT FLOW CONDITIONS

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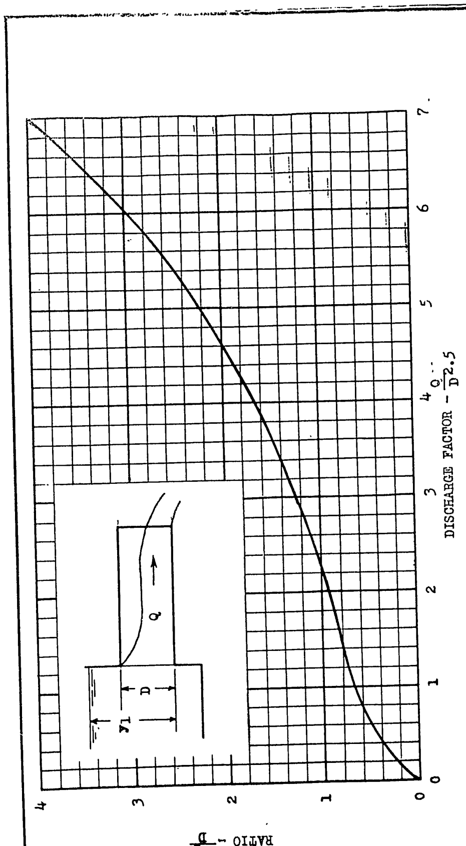

CULVERT STAGE DISCHARGE

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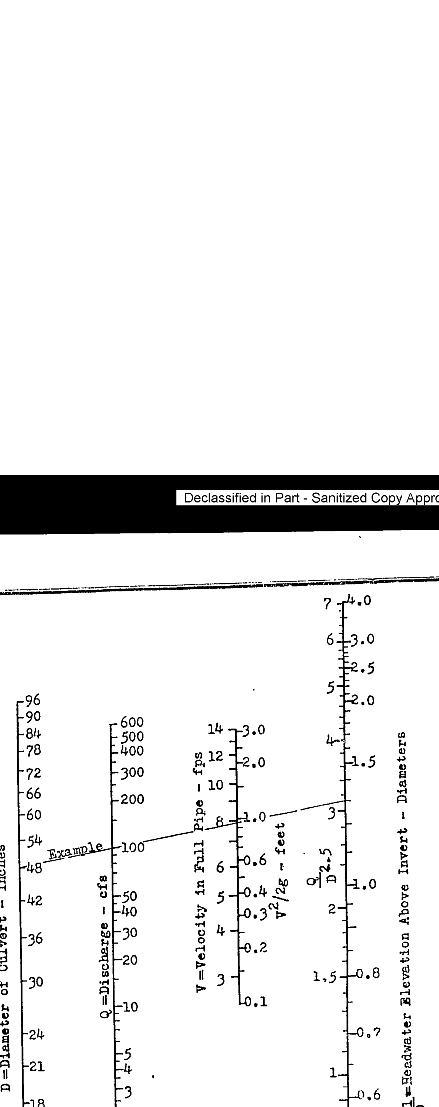
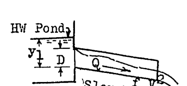
Note:
Stage Discharge Relationship for Control at Inlet Section. Square-Edged Entrance to Circular Pipe Culvert.



EXAMPLE
48" diameter culvert
Q = 100 cfs
Read for D=48" & Q=100 cfs
$y_1/D=1.3$ $y_1=1.3 \cdot 4.0=5.2$ft.



Note:
For free flow circular pipe with square edged entrance and control at inlet.

CIRCULAR CULVERT NOMOGRAPH

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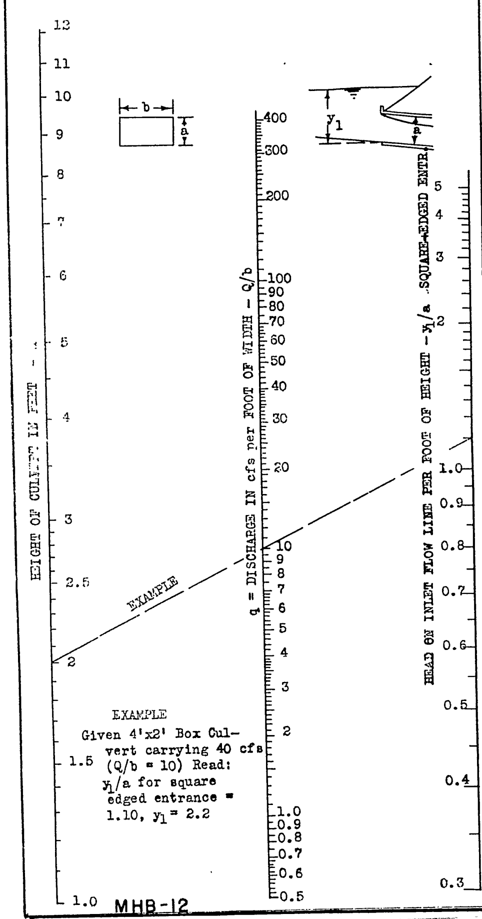
Prepared by ____________ Date __________
Drawn by ____________



EXAMPLE
Given 4'x2' Box Culvert carrying 40 cfs (Q/b = 10) Read: y_1/a for square edged entrance = 1.10, y_1 = 2.2

NOTE: For free flow rectangular culvert with square-edged entrance and control at inlet.

RECTANGULAR CULVERT NOMOGRAPH

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DETERMINATION OF THE DISCHARGE RATING CURVE
FOR A 48" CULVERT

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-
(6) Assumed physical data for the culvert problem.
(7) The normal discharge for the culvert flowing full was computed by methods described in Par. 71. The conveyance of a 4-foot diameter circular pipe with a roughness factor of 0.013 was computed as follows:

$$k' = \frac{1.486}{n} A R^{2/3} \qquad A = 12.57 \text{ ft}^2; \quad R = \frac{D}{4} = 1$$

$$= \frac{1.486}{0.013} 12.57 (1)^{2/3}$$

$$= 1436$$

The normal discharge would be computed as follows:

$$Q = k' (S_o)^{0.5} \qquad S_o = 0.01$$

$$= 1436 (0.01)^{0.5}$$

$$= 143.6 \text{ cfs}$$

All discharges greater than 144 cfs. would have a hydraulic gradient greater than the bottom slope and would cause the culvert to flow full.

HEAD DISCHARGE COMPUTATIONS
FREE FLOW

(8) The discharges were assumed as shown in Col. 1, Table I. The diameter of the culvert in feet was raised to the five-halves power and divided into the discharges of Col. 1 and listed in Col. 2. Entering the curve on Plate 205 with the values of Col. 2, the ratio of y_1/D was obtained and entered in Col. 3. The depth of water above the invert of the culvert for each discharge was computed as the product of the culvert diameter and the ratios of Col. 3, and entered in Col. 4. An alternate method of computing the depth would be to use the nomograph shown on Plate 206. The ratio y_1/D would be determined by extending a straight line connecting the diameter of the culvert and the discharge to intersect the y_1/D scale. For each value of discharge in Col. 1 and a 48" diameter, the value of y_1/D would be read on the nomograph and entered in Col. 3. The depth of water over the invert was then determined as the product of the ratio y_1/D and the diameter of the culvert.

HEAD DISCHARGE COMPUTATIONS
OUTLET SUBMERGED

(9) The head loss through the culvert flowing full and with the outlet submerged was equal to the sum of the exit velocity head, the entrance head loss, and the friction head loss. The friction head was computed by the Darcy-Weisbach formula:

$$h_f = f \frac{L}{D} \frac{V^2}{2g}$$

where f = Darcy-Weisbach resistance coefficient which is equivalent in the Manning equation to $190\ n^2/D^{1/3}$

$$f = \frac{190 \times 0.013^2}{4.0^{1/3}} = 0.020$$

The friction head loss through the culvert was equal to:

$$h_f = (0.020 \times 50/4) \frac{V^2}{2g}$$

$$= 0.25 \frac{V^2}{2g}$$

The entrance loss coefficient of $0.5\ V^2/2g$ for a square edged entrance was taken from the values given in Par. 18. The total head required for each discharge was equal to:

$$h_d = (1 + 0.50 + 0.25) \frac{V^2}{2g}$$

$$h_d = 1.75 \frac{V^2}{2g}$$

(10) The discharges were assumed as shown in Col. 1, Table II. The velocities of Col. 2 were determined from the area of the 48" diameter culvert and the discharges of Col. 1. The velocity head was determined from the velocities of Col. 2 and entered in Col. 3. The total head loss was determined as the product of 1.75 and the velocity heads of Col. 3, and entered in Col. 4.

HEAD DISCHARGE COMPUTATIONS
CULVERT FULL, OUTLET UNSUBMERGED

(11) The upstream depth of water on the culvert flowing full with the outlet unsubmerged was computed by equation 2-6.

$$y_1 = h_d + Z - a$$

The value of "Z" was computed as 0.6D or 2.4 ft. The value of "a" was given in the initial data of item (4) as 0.5 ft. The head loss was computed from Eq. 2-4 as described in item (9). The upstream depth of water on the culvert for each discharge would be equal to:

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$$y_1 = 1.75\ V^2/2g + 0.6\ (4.0) - 0.5$$

$$y_1 = 1.9 + 1.75\ V^2/2g$$

(12) Assume the discharges given in Col. 1, Table III. The velocity head and head loss for each discharge was determined in the same manner as for Cols. 3 and 4 of Table II and entered in Col. 2 and 3 of Table III. The upstream head was computed as the sum of 1.9 and the heads of Col. 3 and entered in Col. 4, Table III.

DISCHARGE RATING CURVE

(13) Flow Characteristic Curves. Flow characteristic curves were determined, for the discharges of Col. 1, Table IV, as described in the following steps:

1. The 45° zero discharge line was drawn with the origin at -0.5 feet headwater depth (0.5 ft. drop in invert elevation between inlet and outlet).
2. A horizontal discharge line was drawn for the constant headwater depths given in Col. 4, Table I, and with the tailwater depths varying from zero to 4 ft. (equal to the diameter of the pipe).
3. The head loss, for the culvert flowing full and the outlet submerged was plotted above the zero discharge line by an amount equal to the values given in Col. 4, Table II. The head losses were drawn as 45° lines on Plate 208F.
4. The flow lines of steps 2 and 3 were connected by nearly vertical lines at a tailwater depth equal to the culvert diameter.
5. The upstream depth for the culvert flowing full and the outlet unsubmerged, was plotted above the upstream invert elevation by the amount given in Col. 4, Table III.
6. The headwater depths of step 5 were connected to the head loss curves computed in step 3 at a tailwater depth equal to the culvert diameter.

Tailwater Rating Curves. For purposes of illustration four tailwater conditions were assumed as follows:

1. Free flow.
2. Moderate tailwater conditions as represented by tailwater rating curve "A" (see Plate 208F).
3. High tailwater conditions as represented by tailwater rating curve "B".
4. A submerged outlet in a large pool as represented by the horizontal tailwater rating curve "C".

Discharge Rating Curves. The headwater depth for free flow conditions, with discharges less than the maximum normal discharge of 146 cfs., was plotted from Col. 1 and Col. 4 of Table I. The headwater depth for discharges greater than the normal discharge was plotted from Cols. 1 and 4 of Table III.

tem

12)

TABLE III
HEAD-DISCHARGE COMPUTATIONS
CULVERT FULL, OUTLET UNSUBMERGED

Q cfs Col. 1	$V^2/2g$-ft. Col. 2	h feet Col. 3	y_1 feet Col. 4
150	2.21	3.87	5.77
200	3.92	6.88	8.78
225	4.97	8.70	10.60

13)

TABLE IV
DISCHARGE RATING CURVE

Q cfs	Free Flow Conditions	Tailwater Condition A		Tailwater Condition B		Tailwater Condition C	
	Depth y_1 ft	Depth y_2 ft	Depth y_1 ft	Depth y_2 ft	Depth y_1 ft	Depth y_2 ft	Depth y_1 ft
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8
50	3.3	4.0	3.9	7.1	7.0	11.0	10.9
100	5.2	5.0	6.5	8.8	10.0	11.0	12.2
150	5.77	5.5	8.8	9.5	12.8	11.0	14.3
200	8.78	5.9	12.2	9.9	16.2	11.00	17.3
225	10.60	6.0	14.2	10.0	18.1	11.0	19.1

CULVERT EXAMPLE

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$$y_1 = 1.75\ V^2/2g + 0.6\ (4.0) - 0.5$$

$$y_1 = 1.9 + 1.75\ V^2/2g$$

(12) Assume the discharges given in Col. 1, Table III. The velocity head and head loss for each discharge was determined in the same manner as for Cols. 3 and 4 of Table II and entered in Col. 2 and 3 of Table III. The upstream head was computed as the sum of 1.9 and the heads of Col. 3 and entered in Col. 4, Table III.

DISCHARGE RATING CURVE

(13) Flow Characteristic Curves. Flow characteristic curves were determined, for the discharges of Col. 1, Table IV, as described in the following steps:

1. The 45° zero discharge line was drawn with the origin at -0.5 feet headwater depth (0.5 ft. drop in invert elevation between inlet and outlet).
2. A horizontal discharge line was drawn for the constant headwater depths given in Col. 4, Table I, and with the tailwater depths varying from zero to 4 ft. (equal to the diameter of the pipe).
3. The head loss, for the culvert flowing full and the outlet submerged was plotted above the zero discharge line by an amount equal to the values given in Col. 4, Table II. The head losses were drawn as 45° lines on Plate 208F.
4. The flow lines of steps 2 and 3 were connected by nearly vertical lines at a tailwater depth equal to the culvert diameter.
5. The upstream depth for the culvert flowing full and the outlet unsubmerged, was plotted above the upstream invert elevation by the amount given in Col. 4, Table III.
6. The headwater depths of step 5 were connected to the head loss curves computed in step 3 at a tailwater depth equal to the culvert diameter.

Tailwater Rating Curves. For purposes of illustration four tailwater conditions were assumed as follows:

1. Free flow.
2. Moderate tailwater conditions as represented by tailwater rating curve "A" (see Plate 208F).
3. High tailwater conditions as represented by tailwater rating curve "B".
4. A submerged outlet in a large pool as represented by the horizontal tailwater rating curve "C".

Discharge Rating Curves. The headwater depth for free flow conditions, with discharges less than the maximum normal discharge of 146 cfs., was plotted from Col. 1 and Col. 4 of Table I. The headwater depth for discharges greater than the normal discharge was plotted from Cols. 1 and 4 of Table III.

The head required to deliver a discharge between 100 and 150 cfs was uncertain and was plotted as a dotted line. The points were connected by smooth curves giving the discharge rating curve for free flow as shown on Plate 208 F.

The discharge rating curves for the three tailwater conditions were computed as described in the following steps:

1. The tailwater depths were determined from rating curves A, B, and C for each discharge given in Col. 1, Table IV, and entered in Cols. 3, 5, and 7, respectively.
2. With the tailwater depths and discharges determined in step 1 determine the headwater depth from the flow characteristic curves and enter in Cols. 4, 6, and 8, respectively.
3. The headwater depths were plotted against the discharges and smooth curves drawn between the points to give the discharge rating curves for tailwater conditions A, B, and C as shown on Plate 208 F.

DETERMINATION OF THE DISCHARGE RATING

Item

INITIAL DATA

(1) 48-inch diameter concrete culvert.
(2) Length = 50 ft.
(3) Concrete headwall with square-edged ent
(4) Slope of culvert is riverward with 0.5
Direction of flow into the river.
(5) Coefficient of roughness = 0.013
(6) $K_e = 0.50$
(7) Max normal discharge = $k' \sqrt{S_o}$ = 143.6

(8) TABLE I

HEAD-DISCHARGE COMPUTATIONS FRE

Q cfs Col. 1	$Q/D^{2.5}$ Col. 2	
50	1.56	
100	3.13	
150	4.70	
200	6.25	
225	7.03	

(9) Head loss for pipe full and outlet subm

$$h_d = (1 + K_e + f\ \frac{L}{D})\ \frac{V^2}{2g} = 1.75$$

(10) TABLE II

HEAD-DISCHARGE COMPUTATIONS

Q cfs Col. 1	V ft/sec Col. 2	$V^2/2$ ft Col.
50	3.97	0.2
100	7.94	0.9
150	11.9	2.2
200	15.9	3.9
225	17.85	4.9

(11) Head loss for culvert full and outlet u

$$y_1 = h + Z - a$$

$$y_1 = 1.9 + h$$

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The head required to deliver a discharge between 100 and 150 cfs was uncertain and was plotted as a dotted line. The points were connected by smooth curves giving the discharge rating curve for free flow as shown on Plate 208 F.

The discharge rating curves for the three tailwater conditions were computed as described in the following steps:

1. The tailwater depths were determined from rating curves A, B, and C for each discharge given in Col. 1, Table IV, and entered in Cols. 3, 5, and 7, respectively.

2. With the tailwater depths and discharges determined in step 1 determine the headwater depth from the flow characteristic curves and enter in Cols. 4, 6, and 8, respectively.

3. The headwater depths were plotted against the discharges and smooth curves drawn between the points to give the discharge rating curves for tailwater conditions A, B, and C as shown on Plate 208 F.

DETERMINATION OF THE DISCHARGE RATING CURVE FOR A 48" CULVERT

Item

INITIAL DATA

(1) 48-inch diameter concrete culvert.
(2) Length = 50 ft.
(3) Concrete headwall with square-edged entrance conditions to pipe.
(4) Slope of culvert is riverward with 0.5 ft. drop in 50 ft. Direction of flow into the river.
(5) Coefficient of roughness = 0.013
(6) $K_e = 0.50$
(7) Max normal discharge = $k'\sqrt{S_o}$ = 143.6 cfs

(8) TABLE I
HEAD-DISCHARGE COMPUTATIONS FREE FLOW CONDITIONS

Q cfs Col. 1	$Q/D^{2.5}$ Col. 2	y_1/D Col. 3	y_1 ft Col. 4
50	1.56	0.82	3.3
100	3.13	1.3	5.2
150	4.70	2.0	8.0
200	6.25	3.25	13.0
225	7.03	4.0	16.0

(9) Head loss for pipe full and outlet submerged:

$$h_d = (1 + K_e + f\frac{L}{D})\ \frac{V^2}{2g} = 1.75\ \frac{V^2}{2g}$$

(10) TABLE II
HEAD-DISCHARGE COMPUTATIONS OUTLET SUBMERGED

Q cfs Col. 1	V ft/sec Col. 2	$V^2/2g$ ft Col. 3	h_d ft Col. 4
50	3.97	0.24	0.42
100	7.94	0.98	1.72
150	11.9	2.21	3.87
200	15.9	3.92	6.88
225	17.85	4.97	8.70

(11) Head loss for culvert full and outlet unsubmerged:

$$y_1 = h + Z - a$$

$$y_1 = 1.9 + h$$

Item

(12) TABLE III
HEAD-DISCHARGE COMPUTATIONS
CULVERT FULL, OUTLET UNSUBMERGED

Q cfs Col. 1	$V^2/2g$-ft. Col. 2	h feet Col. 3	y_1 feet Col. 4
150	2.21	3.87	5.77
200	3.92	6.88	8.78
225	4.97	8.70	10.60

(13) TABLE IV
DISCHARGE RATING CURVE

Q cfs	Free Flow Conditions	Tailwater Condition A		Tailwater Condition B		Tailwater Condition C	
	Depth y_1 ft	Depth y_2 ft	Depth y_1 ft	Depth y_2 ft	Depth y_1 ft	Depth y_2 ft	Depth y_1 ft
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8
50	3.3	4.0	3.9	7.1	7.0	11.0	10.9
100	5.2	5.0	6.5	8.8	10.0	11.0	12.2
150	5.77	5.5	8.8	9.5	12.8	11.0	14.3
200	8.78	5.9	12.2	9.9	16.2	11.00	17.3
225	10.60	6.0	14.2	10.0	18.1	11.0	19.1

CULVERT EXAMPLE

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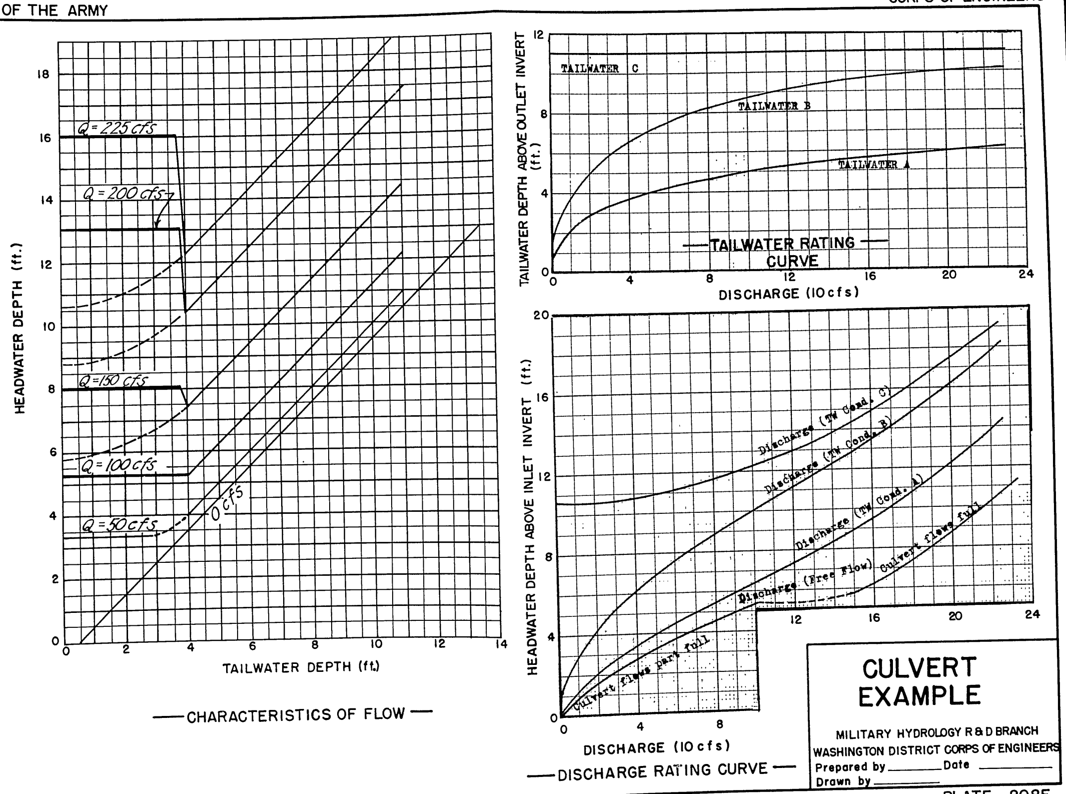

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PLATE 208F

DETERMINATION OF THE DISCHARGE RATING CURVE
FOR A 48" CULVERT WITH ADVERSE SLOPE

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(8) Assumed physical data for culvert problem.

HEAD DISCHARGE COMPUTATIONS
OUTLET SUBMERGED

(9) The head loss through the culvert flowing full and with the outlet submerged was computed as the sum of the exit velocity head, the entrance head loss, and the friction head loss. The head loss for these conditions was computed as explained in items (9) and (10) on Plate 208.

HEAD DISCHARGE COMPUTATIONS
CULVERT FULL, OUTLET UNSUBMERGED

(10) The upstream depth of water on the culvert flowing full with the outlet unsubmerged was computed by equation 2-7:

$$y_1 = h_d + Z + a$$

The value of "Z" was computed as 0.6D or 2.4 ft. The value of "a" was given in item (4) of the initial data as 0.5 ft. The head loss was computed from equation 2-4 as 1.75 $V^2/2g$ and described in item (9), Plate 208.

(11) The discharges were assumed in Col. 1, Table I, Plate 209C. The velocity heads of Col. 2 were determined from the area of the 48" diameter culvert and the discharges of Col. 1. The head loss "h_d" was computed as the product of 1.75 and the velocity heads of Col. 2 and entered in Col. 3. The upstream depth of water of Col. 4 was computed as the sum of the head losses in Col. 3 and 2.9 feet as described in item (10).

DISCHARGE RATING CURVE

(12) <u>Flow Characteristic Curves</u>. Curves were plotted showing the relationship of the headwater and the tailwater depths for the assumed discharges of Col. 1, Table I, as given in the following steps:

1. The zero discharge curve as shown on Plate 209C, was drawn as a 45° line with the origin at 0.5 feet headwater depth (0.5ft. rise in the invert bottom between inlet and outlet).
2. The head loss for the culvert flowing full with the outlet submerged, was plotted above the zero discharge line by an amount equal to the values given in Col. 4, Table II on Plate 208E.
3. The upstream depths for the culvert flowing full

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and the outlet unsubmerged were plotted from the values given in Col. 4, Table I, Plate 209 C.

4. The headwater depths in step (3) were connected with the head loss in step (2) at a tailwater depth equal to the culvert diameter.

Discharge Rating Curves. The tailwater depths were determined for each discharge of Col. 1, Table I from the tailwater rating curves A, B, and C of Plate 208 F and entered in Cols. 2, 4, and 6, Table II, respectively. Entering the flow characteristic curves of Plate 209 C with the tailwater depths of Cols. 2, 4, and 6, and the discharges of Col. 1, the headwater depths were determined and entered in Cols. 3, 5, and 7, respectively. The headwater depths were plotted against the discharges and smooth curves drawn between the points to give the discharge rating curves for tailwater conditions A, B, and C as shown on Plate 209 C.

DETERMINATION OF THE DISCHARGE RATING CURVE FOR A 48" CULVERT WITH ADVERSE SLOPE

Item

INITIAL DATA

(1) 48-inch diameter concrete culvert
(2) Length = 50.0'
(3) Concrete headwalls with square edged entrance and exit conditions.
(4) Slope of culvert is riverward with 0.5 ft. drop in 50 ft.
(5) Direction of flow is landward from flooded river.
(6) Coefficient of roughness = 0.013
(7) $K_e = 0.50$
(8) Tailwater rating curves as shown on Plate 208E
(9) Head for culvert flowing full and outlet submerged taken from Table II, Plate 208E.
(10) Head for culvert flowing full and outlet unsubmerged.

$$y_1 = h_d + Z + a$$

$$\text{where: } h = (1 + K_e + f\,\tfrac{L}{D})\,\frac{V^2}{2g}$$

$$Z = 0.6D$$

$$a = 0.5 \text{ ft}$$

$$y_1 = (1 + K_e + f\tfrac{L}{D})\,\frac{V^2}{2g} + 2.4 + 0.5$$

$$y_1 = 2.9 + 1.75\,\frac{V^2}{2g}$$

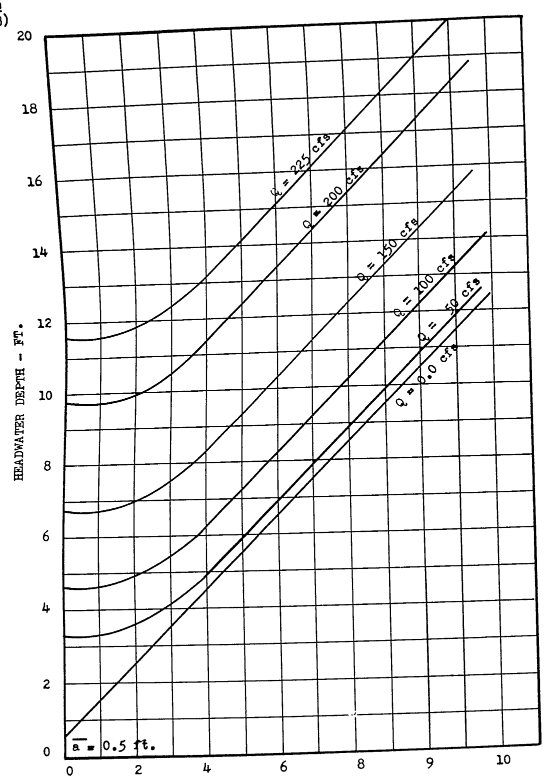
(11) TABLE I
HEAD-DISCHARGE COMPUTATIONS CULVERT FULL, OUTLET UNSUBMERGED

Q cfs Col. 1	$V^2/2g$ ft Col. 2	h_d ft Col. 3	y_1 ft Col. 4
50	0.24	0.42	3.32
100	0.98	1.72	4.62
150	2.21	3.87	6.77
200	3.92	6.88	9.78
225	4.97	8.70	11.60

(12) TABLE II
DISCHARGE RATING CURVES DATA

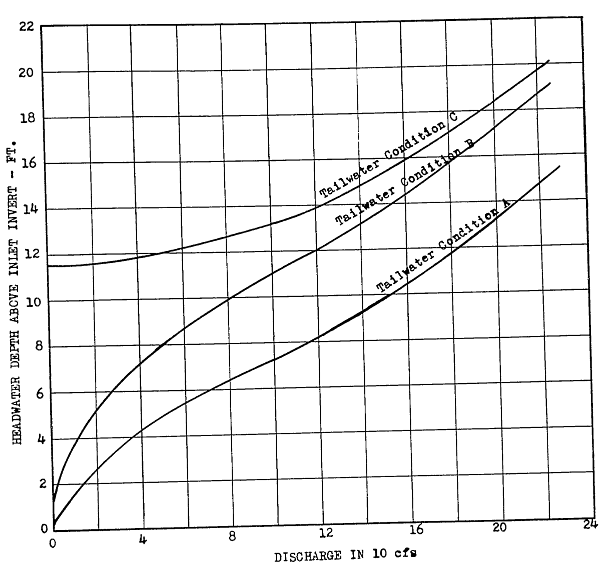
Q cfs	Tailwater Condition A		Tailwater Condition B		Tailwater Condition C	
Col.1	Depth y_2 ft. Col. 2	Depth y_1 ft. Col. 3	Depth y_2 ft. Col. 4	Depth y_1 ft. Col. 5	Depth y_2 ft. Col. 6	Depth y_1 ft. Col. 7
50	4.0	4.9	7.1	8.0	11.0	12.0
100	5.0	7.2	8.8	11.0	11.0	13.2
150	5.5	9.8	9.5	13.8	11.0	15.3
200	5.9	13.2	9.9	17.2	11.0	18.4
225	6.0	15.2	10.0	19.1	11.0	20.1

Item
(13)



CHARACTERISTICS OF FLOW

Item
(14)



DISCHARGE RATING CURVES

ADVERSE SLOPE CULVERT EXAMPLE

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CHAPTER III
WEIRS

SECTION A: BASIC CONSIDERATIONS

22. Definitions. a. A weir is an overflow structure across a stream or waterway used for diverting water or measuring the rate of flow. A measuring weir is a device which usually has a consistent relation between head and discharge. Any barrier across an open channel, over which flow takes place, may be used as a measuring weir. However, continuity of the head-discharge relationship will depend upon the geometry of the weir.

b. The edge or surface over which the water flows is called the crest of the weir. The sheet of water overflowing a weir is called the the nappe. If the tailwater level is below the crest, the weir has a free discharge or free overfall. If the tailwater level is equal to or higher than the crest, the weir is said to be submerged or drowned.

c. When the nappe touches only the upstream edge of the crest, the weir is called sharp-crested or thin-edged. If the nappe touches only the upstream edge of the side of the weir opening, the weir is said to have end contractions. If the sides of the weir are flush with the channel, contraction of the nappe is eliminated and the weir is said to be suppressed.

d. For general usage, the various types of weirs used to measure flow or to serve as reservoir spillways are of such a shape that the head-discharge relationship can be expressed mathematically.

23. Types of Weirs. Weirs normally encountered may be classified according to type of opening such as rectangular, trapezoidal, triangular, and curved weirs; or to profile of crest such as sharp-crested, broad-crested, ogee, triangular, and trapezoidal weirs. Sharp-crested weirs are useful as a means of measuring flowing water. Weirs not sharp-crested are commonly incorporated in hydraulic structures, and though sometimes used to measure water, this is usually a secondary function.

SECTION B: SHARP-CRESTED WEIRS

24. Discharge Capacity. a. In computing the discharge of a particular weir, many conditions and relations must be considered, among the most important of which are:

(1) the ratio of head on the weir to the height of weir
(2) the ratio of the flow area of the approach channel to that at the weir
(3) the effect of the profile shape of the weir; and
(4) the submergence of the weir crest.

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b. The discharge over a sharp-crested weir is computed as follows:

$$Q = 5.347C_qLh^{1.5} \qquad (3\text{-}1)$$

where
Q = discharge in cfs
L = effective length of weir crest
h = head on the weir
C_q = a variable coefficient of discharge

25. <u>Discharge Coefficient</u>. a. The discharge coefficient of a sharp-crested weir is a function of the ratio of weir height to the head on the weir. Rehbock /1/ determined the discharge coefficient of a sharp-crested weir to be:

$$C_q = 0.605 + 0.08\frac{h}{z} + = \frac{1}{305h} \qquad (3\text{-}2)$$

where
C_q = coefficient of discharge
h = head on the weir in feet
z = height of weir in feet

b. The term 0.08 h/z embodies the effect of geometry on the contraction and velocity of approach, and the dimensional term at the end reflects the viscus or capillary influence at very low heads. Neglecting the capillary and viscus influences, the discharge coefficient would be written as:

$$C_q = 0.611 + 0.08\frac{h}{z} \qquad (3\text{-}3)$$

c. The discharge coefficient was plotted as a function of the ratio of the head on the weir to the weir height, and is shown on Plate 301.

26. <u>Nappe Profiles</u>. The upper and lower nappe profiles of a sharp-crested weir are a function of the ratio of the weir height to the head on the weir. Experiments /2/ have been made to determine the coordinates of the nappe profiles and are plotted as dimensionless ratios as shown on Plate 301.

27. <u>Velocity of Approach</u>. a. The approach velocity of the water in the channel above the weir, will affect the discharge of all types of weirs. The velocities throughout the cross section of the approach channel are not uniform, being greater near the center of the stream and just beneath the surface. The velocity head (h_v) is considered to be the head necessary to produce a velocity equal to the mean velocity (Q/A) in the approach channel. The effect of a weir, with or without end contractions, is to cut off that portion of the stream moving with the slower velocities (see Fig. 1, Plate 302). The result

is that the mean velocity for that portion of the flow immediately upstream from and above the weir crest may be much greater than the mean for the entire approach channel. Because this central portion of the flow has a greater effect in increasing the flow through the weir than that which approaches the plane of the weir in a more oblique direction, the effective velocity head (ah_v) is greater than (h_v) and (a) is greater than unity. The total head on the weir then becomes ($h + ah_v$) and

$$Q = 5.35C_qL(h + ah_v)^{1.5} \qquad (3\text{-}4)$$

b. The following values of "a" were calculated from the results of the Francis experiments /1/:

a = 1.5 for suppressed weirs and
a = 2.05 for contracted weirs.

c. The more irregular the velocities the greater the value of "a". As the ratio of the head to height of weir decreases, the value of "a" decreases.

d. Including the effects of velocity of approach but assuming uniform velocities in the channel of approach, the theoretical weir formula is:

$$Q = 5.35C_qL(h + h_v)^{1.5} - h_v^{1.5} \qquad (3\text{-}5)$$

e. Unless the velocity of approach is relative high, the last term ($h_v^{1.5}$) may be omitted and the equation used in the following form:

$$Q = 5.35C_qL(h + h_v)^{1.5} = 5.35C_qLH^{1.5} \qquad (3\text{-}6)$$

f. The selection of Eq. 3-5 or Eq. 3-6 for use in the solution of problems involving the velocity of approach must be made after considering the accuracy desired as well as the relation of the velocity of the approaching stream to the height of the weir. For a low weir having a high velocity of approach, (h_v) is large relative to (h) and Eq. 3-5 should be used.

28. <u>Crest Contraction</u>. a. The vertical contraction at the crest (fig. 1b, Plate 302) is usually the principal source of variation in the discharge over the different types of weirs. This comprises two factors, the curve of the upper surface of the falling sheet, and the contraction of the under surface of the falling sheet at the crest edge. The contraction of the under surface varies with the cross section of the weir crest. By experiment /3/, the ratio of the actual area to the theoretical area of the falling sheet for a sharp crest was found to be about 0.623. Applying this ratio, as the coefficient of discharge, to the constant 5.347, the new constant becomes 0.623 x 5.347 = 3.33. The discharge over a sharp-crested rectangular suppressed weir would be computed by the equation:

$$Q = 3.33LH^{1.5} \qquad (3\text{-}7)$$

which is known as the Francis formula.

b. The crest contraction of a sharp-crested weir is complete when the ratio of height of weir (z) to the total head on the weir (H) is 3 or more. The maximum vertical rise of the lower nappe above a sharp-crested weir is approximately equal to 0.112 H.

29. <u>End Contractions</u>. a. If the sides of the notch or weir have sharp upstream edges so that the nappe is contracted in width, the weir is said to have end contractions. For full end contractions, the ends of the weir should not be closer to the sides of the channel than 2.5 times the head on the weir.

b. End contractions have the effect of reducing the effective length of a weir. It has been found by experiment /3/ that a sharp edge is equivalent to reducing the length by about 0.1 H: The following formula has been used to determine the effective length:

$$L = L' - 0.1\ NH \tag{3-8}$$

where
L' = the gross length of the weir
N = the number of contractions
H = the total head on the weir in feet

30. <u>Inclined Weirs</u>. An upstream inclination of a sharp-crested weir (Fig. 2a, Plate 302) reduces, while a downstream inclination (Fig. 2b, Plate 302) increases, the rate of discharge as indicated in the following tabulation /3/:

TABLE 1

	Upstream			Vertical	Downstream				
Slope Hor/Vert.	1:1	2:3	1:3	0	1:3	2:3	1:1	2:1	4:1
Relative Q	.93	.94	.96	1.00	1.04	1.07	1.10	1.12	1.09
Const. in Eq. 3-7	3.10	3.13	3.20	3.33	3.46	3.56	3.66	4.00	3.63

31. <u>Curved Weirs</u>. a. Weirs curved in plan may be considered as straight for all practical purposes if the water approaching and leaving the weir travels in radial lines. The arc length would be used for (L) in the basic weir formula. If, however, a curved weir of short radius is placed in a confined channel in which the approaching water is not free to travel radially to the weir, the chord length or at least some reduction in arc length would be used.

b. The extreme case of a curved weir is found where the crest of the weir forms a complete circle. In this case, the effective length of the weir is the circumference of a circle at the control point of the weir. Discussion for conditions of flow other than true weir discharge will be found under Morning Glory Spillways in Chapter VI.

32. <u>Weirs with Crests Not Level</u>. a. If the crest of the weir is only slightly inclined, (Fig. 3, Plate 302), the discharge would be approximated by use of the average head in the basic weir equation. If the variation in head is considerable, the following equation would be used:

$$Q = \frac{2}{5}C_q \left[\frac{\text{[illegible]}}{h_2 - h_1}\right](h_2^{2.5} - h_1^{2.5}) \tag{3-9}$$

b. The coefficient (C_q) in the above equation is the usual weir coefficient, its numerical value depending upon the type of weir.

c. Where the ratio of h_1/h_2 is greater than 0.8, the difference resulting from the use of the two formulas will not be greater than 5 percent. If there are end contractions, the effective length should be used as described in Par. 29.

33. <u>Submerged Weirs</u>. a. Where the water on the downstream side of a weir rises to a level above the weir crest, the weir is said to be submerged. The discharge of a submerged weir (see Fig. 4, Plate 302) may be regarded theoretically as composed of two parts: (1) that through the upper part (h - N), which may be considered as free discharge, and (2) that through the lower part (N) which may be considered as flow through a submerged orifice. Herschell /3/ developed a submerged weir formula which is dependent only upon the difference in elevation between the upstream and downstream water surfaces. This formula for a sharp-crested submerged weir is:

$$Q = 3.33L(C_s h)^{1.5} \tag{3-10}$$

where:
Q = discharge in cfs
L = the effective length of weir crest in ft.
h = the upstream head on the weir crest in ft.
C_s = Herschel's submerged weir coefficient.

b. The coefficient C_s is dependent upon the ratio of the submerged head (N) to the upstream head (h), and is shown on Plate 303.

34. <u>V-Notch Weirs</u>. a. The measurement of a small discharge is normally made over a sharp-crested V-notch weir. (Fig. 5, Plate 302). The discharge over the sharp-crested V-notch weir is computed by the equation:

$$Q = \frac{8}{15}C_q s(2g)^{0.5}h^{2.5} \tag{3-11}$$

where:
h = the head on the weir in ft.
s = the side slope of notch or L/2h
C_q = the coefficient of discharge
L = the width of the notch at height h

b. The coefficient of discharge /3/ for a 90° V-notched weir, where (s) is unity, was found by experiment to be 0.593. The discharge is computed by the equation:

$$Q = 2.54\ h^{2.5} \qquad (3\text{-}12)$$

c. The discharge for a V-notch weir with 2:1 side slopes and a coefficient of discharge of 0.593 is computed by the equation:

$$Q = 5.07\ h^{2.5} \qquad (3\text{-}13)$$

35. Trapezoidal Weirs. a. The discharge through a trapezoidal weir (Fig. 6, Plate 302) may be considered as the sum of the discharges through a suppressed rectangular notch and a triangular or V-notch. The discharge over a sharp-crested trapezoidal weir, with side slopes of 1 on s is computed by the equation

$$Q = \frac{2}{3}C_1\ Lh^{1.5}\ (2g)^{0.5} + \frac{8}{15}\ C_2 sh^{2.5}(2g)^{0.5} \qquad (3\text{-}14)$$

b. The discharge over a sharp-crested trapezoidal weir with side slopes of 1 on 1 is:

$$Q = 3.33\ Lh^{1.5} + 2.54\ h^{2.5} \qquad (3\text{-}15)$$

c. The Cippoletti weir is a trapezoidal weir having side slopes of 1 on 1/4 which is approximately the slope required to discharge a quantity of water through the triangular portion of the weir equal to the decrease in discharge through the rectangular notch due to end contractions. The advantage of this type of weir is that no correction in length is necessary for end contractions.

d. The discharges over a sharp-crested trapezoidal weir with 1 on 1/4 side slopes, based upon Cippoletti's experiments is:

$$Q = 3.367\ Lh^{1.5} \qquad (3\text{-}16)$$

36. Side Weirs. a. A side weir (Fig. 7, Plate 302) consists of a rectangular notch in the side of a canal with the axis parallel to the line of flow. Side weirs are used principally to reduce the water surface elevation, in a flume or canal, to the elevation of the lip. Experiments /5/ show that the necessary length of weir to accomplish this is given by the formula

$$L = 29.1\ b_w^{1.4} H^{0.513} \qquad (3\text{-}17)$$

where $H = h + h_v$,

b_w = the water surface width of the main channel

b. The discharge over the weir was found by experiment to be

$$Q = 1.674\ b_w L^{0.72}\ H^{1.645} \qquad (3\text{-}18)$$

SECTION C. WEIRS NOT SHARP-CRESTED

37. Nappe Form a. The quantity of water which will pass over a weir, under a given head, depends to a great extent upon the shape of the crest, and the form the nappe takes in passing over the crest. For each modification of the nappe form, there is a corresponding change in the relation between head and discharge. In general, the nappe /3/ may undergo any of the following modifications as the head is varied: discharge freely by touching only the upstream edge, adhere to top of crest, adhere to downstream face of crest, adhere to both top and downstream face, remain detached but become wetted underneath, adhere to top but remain detached from face, and become wetted underneath causing a partial vacuum. The nappe may undergo several of these modifications in succession as the head is varied. The modifications of nappe form are usually confined to comparatively low heads.

b. As an example of this relation between nappe and discharge, the following coefficients in Eq. 3-7 apply to a thin-edged weir 2.46 ft. high discharging under a head of 0.656 feet. These coefficients are based on the total head and include the effect of the velocity of approach.

Condition of Nappe	Coefficient
Free discharge, full aeration	3.47
Nappe depressed, partial vacuum underneath	3.69
Nappe wetted underneath, downstream water level, 0.42 feet below crest	3.99
Nappe adhering to downstream face of weir	4.45

c. For most forms of weirs of irregular section, the departure in weir coefficient from that applying to a sharp-crested weir results from some permanent modification of the nappe form. Weirs with sloping upstream faces reduce the amount of crest contraction, broad-crested weirs cause adherence of the nappe to the crest, and aprons cause permanent adherence of the nappe to the downstream face.

38. Free Overfall. The critical depth /6/ occurs a short distance upstream from the brink of a free overfall on a channel of mild slope (Fig. 1, Plate 304). The critical depth location is approximately 12 y_c upstream from the brink for a channel with a level bed, the distance increasing as the slope of the channel increases. The depth at the brink is $0.715y_c$ on channels with bottom slopes less than critical. The unit discharge over an overfall is computed by the following equation:

$$q = (gy_c^{\,3})^{0.5} = 9.4\ y_b^{\,1.5} \qquad (3\text{-}19)$$

where

q = discharge per ft. of width
y_c = critical depth in ft.
y_b = depth at the brink of the overfall in ft.

39. Broad-Crested Weirs. a. A weir (Fig. 2, Plate 304), approximately rectangular in section with a nearly flat upper surface, is termed a broad-crested weir. If the broad-crested weir has a sharp-cornered upstream face, the lower nappe profile will be contracted. Surface contraction begins at a point slightly

upstream from the weir and continues to curve downward, passing through a point of inflection, and becomming tangent to a plane approximately parallel to the crest a short distance below the upstream edge of the weir.

b. The flow over a broad-crested weir occurs at critical depth and the discharge per unit width for a weir with a rectangular flow section is:

$$q = (gy_c{}^3)^{0.5} \qquad (3\text{-}20)$$

where

q = the discharge per foot of width in cfs
g = acceleration of gravity
y_c= the critical depth in feet

The critical depth is:

$$y_c = 2/3\ H \qquad (3\text{-}21)$$

where

H = the total head on the weir in feet.

c. The theoretical discharge over a broad-crested weir with a perfectly rounded upstream edge is:

$$Q = 3.09\ LH^{1.5} \qquad (3\text{-}22)$$

d. The discharge coefficient as determined in model and prototype testing is lower. The coefficient of discharge is a function of the degree of rounding of the upstream corner of the crest, the amount the weir crest slopes in the direction of flow, and the ratio of the head on the weir to the critical depth.

40. Ogee Weir. a. A comparison of the sharp-crested and ogee weirs is indicated in Fig. 4, Plate 304. By experiment /7/ the rise in the lower surface of the nappe over a sharp-crested weir has been found to be approximately 0.112 h, the distance out to the point of maximum rise approximately 0.25 h, and the drop in the water surface from still water to a point vertically above the high point of the under side of the nappe is approximately 0.22 h. Assuming that the area beneath the lower surface of the nappe is filled with concrete, the discharge (Q) remains unchanged. The crest of the weir, however, is at an elevation 0.888h = H below water surface. Then h = 1.126 H. Substituting in Eq. 3-7:

$$Q = 3.33\ L\ (1.126H)^{1.5}$$
$$Q = 3.97\ LH^{1.5} \qquad (3\text{-}23)$$

therefore, for the ogee weir, the theoretical coefficient C_d = 3.97.

b. This coefficient will apply for that head (H) on the ogee weir at which the concrete section just fits the lower surface on the nappe of the sharp-crested weir for head (h). The results of numerous experiments

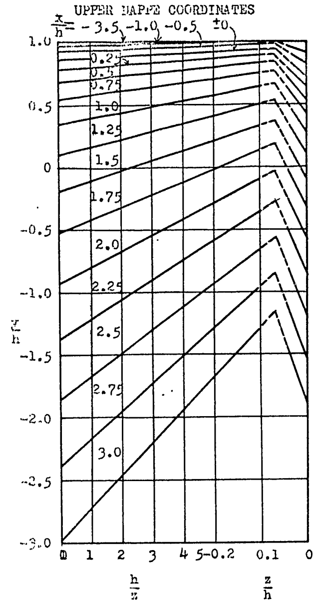
indicate values of C_d are seldom higher than 3.8. Using a value of C_d of 3.65 in Eq. 3-23, will usually result in discharges that are correct within a range of 10 percent. Factors causing the variation of C_d are discussed in the section of Ogee Spillways in Chapter VI.

41. Discharge Coefficient. Experiments have been perfomed to determine the coefficients of discharge for many types of broad-crested weirs. The more common types with their experimental coefficients are given in Plates 305 to 310, inclusive.

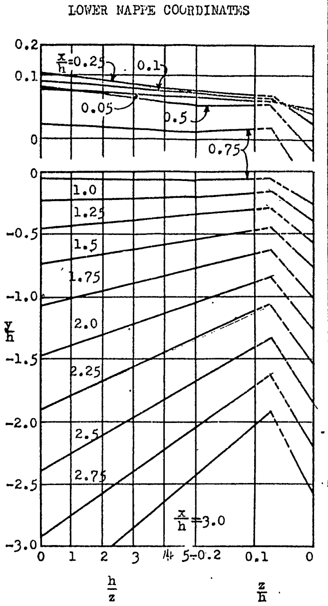
42. References

/1/ Rehbock, Theodore. "Wassermessung mit Scharfkantegen Ueberfallwehren." Zeitschrift des Vereins Deutscher Ingenieure, Vol. 73, May 15, 1929.
/2/ Rouse, Hunter. Fluid Mechanics for Hydraulic Engineers. New York: McGraw-Hill Book Co., 1938.
/3/ Horton, R.E. "Weir Experiments, Coefficients and Formulas." U.S. Geological Survey Water Supply Paper No. 200, Department of the Interior. Revision of Paper No. 150, Washington: U.S. Government Printing Office, 1907.
/4/ "Hydraulic and Excavation Tables," Bureau of Reclamation, Department of the Interior, Washington: U.S. Government Printing Office, 1944.
/5/ Coleman and Smith. "Selected Engineering Papers," No. 6, 1923. Proc. Inst. Civ. Engr.
/6/ Rouse, Hunter. "Discharge Characteristics of the Free Overfall," Civil Engineering, Vol. 6, No. 4, April 1936.
/7/ Bazin, H. Expériences Nouvelles sur l' écoulement en déversoir: Ann. Ponts et Chaussées, Mém. et Doc. 1898, 2 me Trimestre. Translated Marichal and Troutwine in Proc. Eng. Club, Phila., Vol. 7,9, and 10.

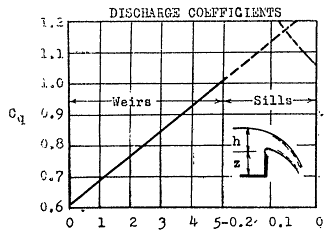
UPPER NAPPE COORDINATES



LOWER NAPPE COORDINATES



DISCHARGE COEFFICIENTS



WEIR DISCHARGE COEFFICIENT & NAPPE COORDINATES

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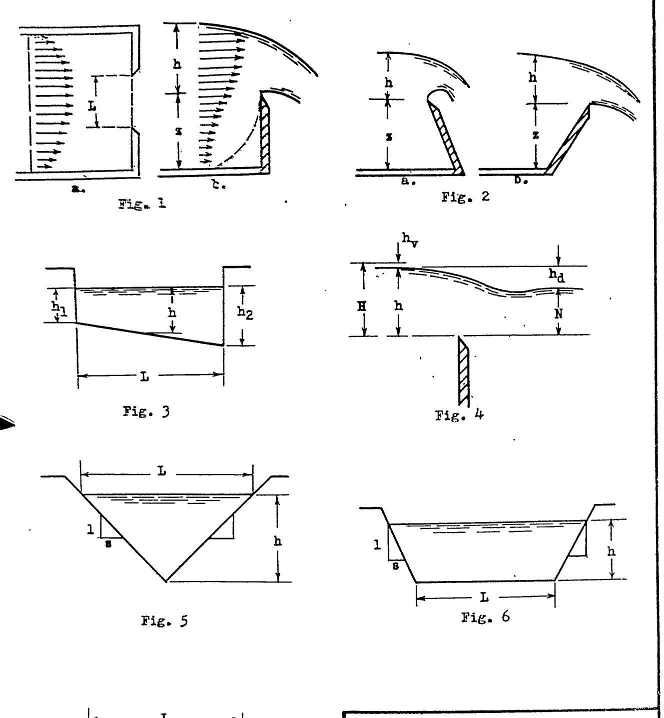

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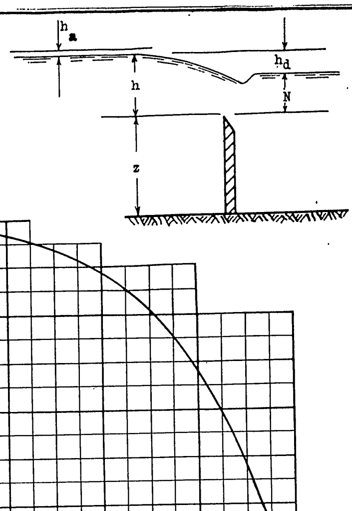
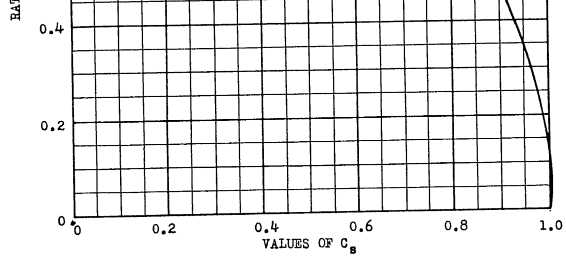
$Q = 3.33\ L\ (C_s h)^{1.5}$





HERSCHEL'S SUBMERGED WEIR COEFFICIENT

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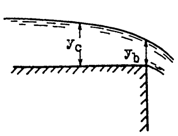

Fig. 1
FREE OVERFALL

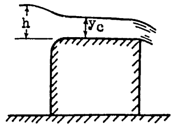

Fig. 2
BROAD-CRESTED WEIR

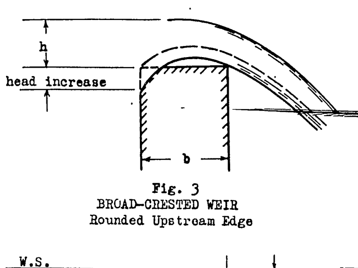

Fig. 3
BROAD-CRESTED WEIR
Rounded Upstream Edge

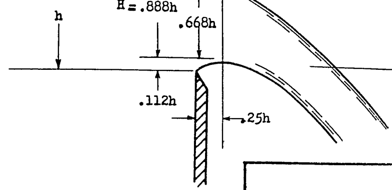

Fig. 4
OGEE WEIR

WEIR DATA

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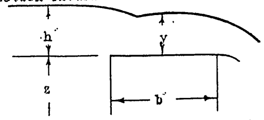
TABLE I

BROAD-CRESTED WEIR - FACES VERTICAL *
Correction should be made for velocity of approach



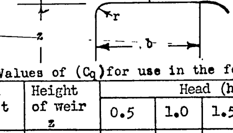
Values of (C_q) for use in Formula $Q = C_q L h^{1.5}$

Head "h" in ft.	Breadth of Crest (b) in Feet								
	0.5	0.75	1.0	2.0	3.0	4.0	5.0	10.0	15.0
0.5	3.00	2.85	2.74	2.61	2.63	2.62	2.60	2.63	2.70
1.0	3.32	3.14	2.98	2.66	2.65	2.67	2.68	2.68	2.63
1.5	3.32	3.28	3.24	2.83	2.66	2.65	2.64	2.64	2.63
2.0	3.32	3.31	3.30	2.85	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.07	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.20	2.92	2.73	2.66	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
6.0	3.32	3.32	3.32	3.32	3.32	3.32	2.90	2.64	2.63

* "Handbook of Hydraulics", Horace W. King.

TABLE II

BROAD-CRESTED WEIR - ROUNDED UPSTREAM CORNER *
Correction should be made for velocity of approach



Values of (C_q) for use in the formula $Q = C_q L h^{1.5}$

Radius of curve r	Breadth of crest b	Height of weir z	Head (h) in Feet						
			0.5	1.0	1.5	2.0	3.0	4.0	5.0
0.33	2.62	2.46	2.95	3.01	3.04				
0.33	6.56	2.46	2.76	2.89	2.92				
0.33	2.62	4.57	2.75	2.83	2.92	3.0	3.17	3.34	3.50
0.33	6.56	4.56	2.83	2.83	2.83	2.82	2.82	2.82	2.81

* U. S. Geological Survey, Water Supply Paper #200

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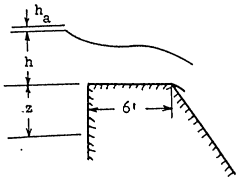
TABLE III, BROAD CRESTED WEIR

Downstream Face Sloping - Upstream Face Vertical

No correction should be made for velocity of approach



Experiments on Merced Falls Dam*

Values of (C_q) for use in the formula $Q = C_qLh^{1.5}$

h	1	2	3	4	5	6
Cq	2.50	3.00	3.35	3.62	3.85	4.04

The above values of C_q were computed from actual measurements of h and Q.

No correction was made for the velocity of approach which was relatively high.

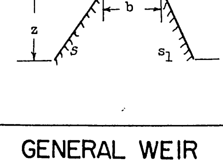
TABLE IV

TRAPEZOIDAL-CRESTED WEIRS **

Correction should be made for the velocity of Approach

Values of (C_q) for use in the Formula $Q = C_qLh^{1.5}$

Slopes		Crest	Head in Feet							
Upstream s	Downstream s_1	Width b	0.5	1.0	1.5	2.0	2.5	3.0	4.0	5.0
2:1	2:1	0.67			3.58	3.56	3.57	3.58	3.62	3.68
2:1	5:1	0.33			3.59	3.53	3.48	3.44	3.48	3.57
2:1	Vert.	0.33		3.85	3.82	3.79	3.77	3.75	3.70	3.64
2:1	Vert.	0.66		3.41	3.57	3.65	3.70	3.72	3.73	3.73
3:1	Vert.	0.66			3.57	3.57	3.57	3.57	3.57	3.57
4:1	Vert.	0.66			3.48	3.48	3.48	3.48	3.48	3.48
5:1	Vert.	0.66			3.39	3.39	3.39	3.39	3.39	3.39
1:1	2:1	0.67	3.02	3.42	3.65					
1:2	1:1	0.66	3.02	3.52	3.82					
1:2	2:1	0.66	2.92	3.38	3.61					
1:2	3:1	0.66	2.91	3.26	3.45					
1:2	4:1	0.66	2.88	3.21	3.35					
1:2	5:1	0.66	2.88	3.17	3.26					
1:3	2:1	0.67	2.87	3.34	3.55					
2:1	2:1	0.67	3.13	3.43	3.61					
1:2	2:1	1.32	2.80	2.98	3.22					
1:2	4:1	1.32	2.82	2.94	3.10					
1:2	6:1	1.32	2.80	2.93	3.08					
1:1	1:1	0	4..4	4.08	3.75					
1:1	2:1	0	3.77	3.85	3.84					
1:1	3:1	0	3.48	3.48	3.46					
2:1	2:1	0	3.81	3.87	3.87					
1:2	2:1	0	3.69	3.74	3.71					
1:3	2:1	0	3.67	3.69	3.66					



For the first seven experiments liste , the height of weir used was 4.9 feet. For all others the height was 2.46 feet.

* "ENGINEERING NEWS." Sept. 29, 1910, pg. 321

** U.S. GEOLOGICAL SURVEY, Water Supply Paper #200.

GENERAL WEIR DATA

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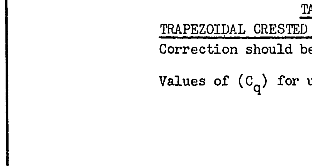
TABLE V

TRIANGULAR CRESTED WEIRS - VERTICAL UPSTREAM FACE *

Correction should be made for velocity of approach

Values of (C_q) for use in the formula $Q = C_qLh^{1.5}$



Slope Hor. to Vert.	z	Av. Cq
1:1	2.46	3.85
2:1	2.46	3.50
5:1	2.46	3.13
10:1	2.46	2.91
2:1	1.64	3.57
3:1	1.64	3.40

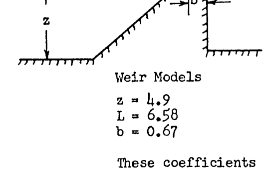
Coefficients remain practically constant, for varying heads, for each slope, until the nappe jumps free from the downstream face, in which case it becomes a sharp crested weir. Values are fairly accurate for heads varying from .7 to 1.5 feet. Test model length 6.6 feet. Lower nappe not aerated.

TABLE VI

TRAPEZOIDAL CRESTED WEIR - VERTICAL DOWNSTREAM FACE *

Correction should be made for velocity of approach

Values of (C_q) for use in the formula $Q = C_qLh^{1.5}$



Slope	Cq
1:1	3.7
2:1	3.7
3:1	3.58
4:1	3.49
5:1	3.39
6:1	3.33
7:1	2.98

Weir Models

z = 4.9
L = 6.58
b = 0.67

These coefficients are applicable for heads varying from 2 to 5 feet.

* U.S. GEOLOGICAL SURVEY, WS Paper #200

GENERAL WEIR DATA

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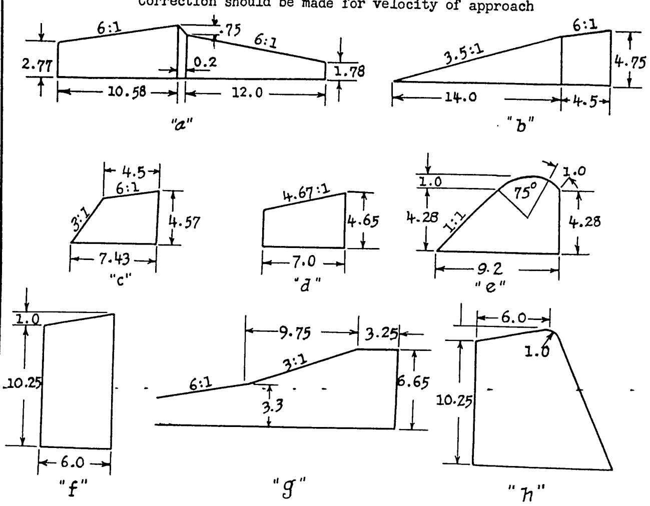
TABLE I

WEIRS OF IRREGULAR CROSS SECTION*

Correction should be made for velocity of approach



Values of (C_q) for use in the formula $Q = C_qLh^{1.5}$

Fig.	Length	Head (h) in feet								
		0.5	1.0	1.5	2.0	2.5	3.0	4.0	5.0	6.0
a	6.6	3.0	3.13	3.22	3.22	3.22	3.22	3.22	3.22	3.22
b	6.6			3.41	3.35	3.30	3.33	3.38	3.38	
c	6.6	3.45	3.47	3.46	3.41	3.35	3.32	3.37	3.46	
d	6.6				3.50	3.44	3.39	3.38	3.41	3.42
e	6.6	3.25	3.28	3.29	3.32	3.39	3.46	3.59	3.65	3.68
f	16	3.53	3.54	3.55	3.50	3.35	3.27	3.25		
g	16	3.28	3.50	3.55	3.52	3.35	3.30	3.30		
h	16	3.13	3.15	3.10	3.15	3.20	3.25	3.36		

* U.S.GEOLOGICAL SURVEY, WS Paper #200

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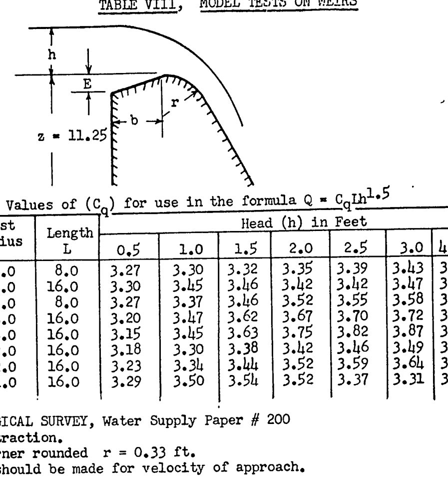
TABLE VIII, MODEL TESTS ON WEIRS*



Values of (C_q) for use in the formula $Q = C_qLh^{1.5}$

Width b	Rise E	Crest Radius r	Length L	Head (h) in Feet							
				0.5	1.0	1.5	2.0	2.5	3.0	4.0	5.0
** 3.0	0.75	3.0	8.0	3.27	3.30	3.32	3.35	3.39	3.43	3.55	3.72
3.0	0.75	3.0	16.0	3.30	3.45	3.46	3.42	3.42	3.47	3.52	
** 3.0	1.50	3.0	8.0	3.27	3.37	3.46	3.52	3.55	3.58	3.68	3.83
3.0	1.50	3.0	16.0	3.20	3.47	3.62	3.67	3.70	3.72	3.74	
3.0	2.88	3.0	16.0	3.15	3.45	3.63	3.75	3.82	3.87	3.88	
4.5	1.00	2.0	16.0	3.18	3.30	3.38	3.42	3.46	3.49	3.55	
*** 4.5	1.00	2.0	16.0	3.23	3.34	3.44	3.52	3.59	3.64	3.70	
6.0	1.00	1.0	16.0	3.29	3.50	3.54	3.52	3.37	3.31	3.30	

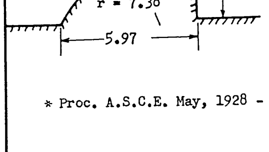
* U. S. GEOLOGICAL SURVEY, Water Supply Paper # 200
** One end contraction.
*** Upstream corner rounded r = 0.33 ft.
Correction should be made for velocity of approach.

TABLE IX

WEIR WITH CIRCULAR UPSTREAM FACE*

Correction should be made for velocity of approach

Computed values of (C_q) for use in the formula $Q = C_qLh^{1.5}$



h	0.5	1.0	1.5	2.0	2.5	3.0
C_q	3.03	3.10	3.20	3.27	3.36	3.46

Length of weir 52.5 feet
Tests were made on full size weir

* Proc. A.S.C.E. May, 1928 - P. 1493

GENERAL WEIR DATA

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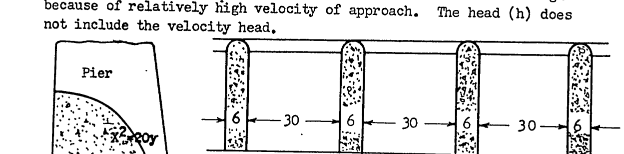
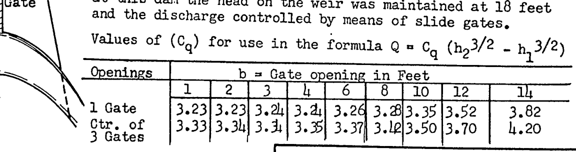
TABLE X, RESULTS OF TESTS - IRREGULAR CRESTS

Values of (C_q) for use in the formula $Q = C_q L h^{1.5}$



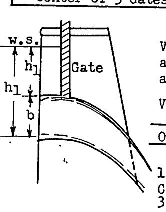
Dam	Head (h) in Feet					
	1	2	3	4	5	6
La Grange *	3.09	3.09	3.09	3.09	3.09	3.09
Austin *	3.2	3.4	3.52	3.63	3.69	3.75
Yakima *	3.11	3.45	3.67	3.83	3.96	4.08

The above values of (C_q) are in some instances believed to be high because of relatively high velocity of approach. The head (h) does not include the velocity head.



Keokuk Dam **

	Head (h) in Feet								
	1	2	3	4	5	6	8	10	12
1 Gate	2.95	3.02	3.13	3.23	3.32	3.41	3.56	3.66	3.73
Center of 3 Gates					3.50	3.55	3.66	3.77	3.88



Wilson Dam ***

Values check the above with reasonable limits. However at this dam the head on the weir was maintained at 18 feet and the discharge controlled by means of slide gates.

Values of (C_q) for use in the formula $Q = C_q\ (h_2^{3/2} - h_1^{3/2})$

Openings	b = Gate opening in Feet								
	1	2	3	4	6	8	10	12	14
1 Gate	3.23	3.23	3.24	3.24	3.26	3.28	3.35	3.52	3.82
Ctr. of 3 Gates	3.33	3.34	3.34	3.35	3.37	3.42	3.50	3.70	4.20

* Eng. News, Sept. 29, 1910
** Trans., A.S.C.E., 1930
*** Trans., A.S.C.E., 1931

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PLATE 310 MHB-12

CHAPTER IV PIPE FLOW

SECTION A: BASIC CONSIDERATIONS

43. Definition. A pipe is a closed conduit which carries water under pressure. Conduits flowing partially full are considered to be open channels and are analyzed in Chapter V.

44. Basic Theory a. Bernoulli's Theorem is the basis of pipe flow formulas. Bernoulli's Equation (equation 1-10), disregarding losses due to friction, entrance, bends, contraction, and expansion (see Fig. 1, Plate 401) is:

$$\frac{V_1^2}{2g} + P_{1/} + Z_1 = \frac{V_1^2}{2g} + P_{2/} + Z_2 = \frac{V_3^2}{2g} + P_{3/} + Z_3 = H \tag{4-1}$$

including these losses (Fig. 2, Plate 401):

$$H = \frac{V_1^2}{2g} + P_{1/} + Z_1 + h_L(A\text{ to }1) = \frac{V_2^2}{2g} + P_{2/} + Z_2 + h_L\ (A\text{ to }2)$$

$$= \frac{V_3^2}{2g} + P_{3/} + Z_3 + h_L\ (A\text{ to }3) = \frac{V_4^2}{2g} + h_L\ (A\text{ to }4) \tag{4-2}$$

b. Where V_1, V_2, V_3, and V_4 are the average velocities in the pipe in the respective sections, and h_L (A to 1), h_L (A to 2), etc., are the losses between (A) and (1), (A) and (2), etc. Then:

$$H = \frac{V_4^2}{2g} + h_L\ (A\text{ to }1) + h_L\ (1\text{ to }2) + h_L\ (2\text{ to }3) + h_L\ (3\text{ to }4) \tag{4-3}$$

or the total head (H) is equal to the velocity head of the issuing stream plus all losses in the line above the outlet.

c. If the total losses are (H_L), then:

$$H = \frac{V_o^2}{2g} + H_L \tag{4-4}$$

or

$$H - H_L = V_o^2/2g \tag{4-5}$$

where V_o is the velocity of the water at its exit. Since Q, the quantity of flow is constant and the areas are known at all sections, the velocity may be determined from Q = AV.

45. Hydraulic and Energy Grade Lines. The hydraulic grade line is a line which would connect the water surfaces in a series of open tubes were placed along a pipe line. The distance from the hydraulic grade line to the center of the pipe at any point measures the pressure head on the pipe. Owing to continuity effects reflected in the velocity head, form losses at changes in section cause abrupt changes in elevation of the hydraulic grade line. Between these abrupt changes, the line slopes downward. The slope of this line is h_f/L, where h_f is the friction head loss which occurs in length L of the pipe. For uniform flow, the slope of this line must also equal the slope of the energy gradient. The energy grade line is a line passing through points which lie above the hydraulic grade line by an amount equal to the velocity head.

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46. Loss of Head. Loss of head occurs in any flow through a pipe. The loss is caused by: (1) "pipe friction" along the straight sections of pipe of uniform diameter and uniform roughness; and (2) changes in velocity or direction of flow (form losses).

SECTION B: FRICTION LOSS

47. The Darcy Weisbach Equation. a. This equation is the basis of most pipe friction calculations and is:

$$h_f = f \frac{L}{D} \frac{V^2}{2g} \qquad (4\text{-}6)$$

where

h_f = loss of head in feet of water
f = the friction factor
L = pipe length in feet
D = diameter in feet

b. The table on Plate 402 shows average values of "f" as given by Fanning /1/ for the turbulent flow of water at natural "cold-water" temperatures in straight smooth pipes. This description probably represents the conditions of new cast-iron pipe, welded-steel pipe, wood pipe made of planed staves, concrete pressure pipe of best quality, and cement-lined steel pipe.

c. Flow in conduits not circular in cross section may be studied by using the relationship between the hydraulic radius and diameter

$$4R = D \qquad (4\text{-}7)$$

d. The above equation is applicable so long as the width-depth ratio does not become excessively large or small.

48. Manning's Formula. a. Friction head loss may be obtained by the use of Manning's Formula in the following form:

$$h_f = \frac{2g\ L\ n^2}{2.21\ R^{4/3}} \frac{V^2}{2g} = K_f \frac{V^2}{2g} \qquad (4\text{-}8)$$

where

L = length of pipe in feet
R = mean hydraulic radius in feet for the reach considered

b. For any given roughness factor, n, (Plate 501)

$$K_f = \frac{C_f\ L}{R^{4/3}} \qquad (4\text{-}9)$$

where

$$C_f = \frac{2gn^2}{2.21}$$

c. Values of C_f corresponding to several "n" values are tabulated:

n	C_f
0.010	0.00291
0.011	0.00352
0.012	0.00419
0.013	0.00492
0.014	0.00571

d. Maning's "n" can be converted to the Darcy-Weisbach "f" factor in circular pipes, by the following equations: /2/

$$f = \frac{184.8\ n^2}{D^{1/3}} \qquad (4\text{-}10)$$

or

$$n = \frac{D^{1/6}}{13.59} (f)^{1/2} \qquad (4\text{-}11)$$

SECTION C: FORM LOSS

49. Definition. Form losses vary roughly as the square of the velocity, and they are commonly expressed by applying variable coefficients to the velocity head. Losses due to changes in velocity and direction are given in the following paragraphs.

50. Loss of Head at Intake Racks (h_r). a. The loss of head through a trash rack would be computed by the following equation

$$h_r = K_r \frac{V^2}{2g} \qquad (4\text{-}12)$$

where

h_r = the loss of head through the trash racks in feet
V = the velocity in the net area of racks and supports
K_r = head loss coefficient through the trash racks /3/
$= 1.45 - 0.45A_r - A_r^2$ (4-13)
A_r is the ratio of net to gross area of racks and supports.

b. For a value of K_r = 0.65 the resulting value of K_r - 0.74, and for a velocity of 2.5 ft. per second in the net area, the loss is 0.072 ft. Twenty-five to fifty percent of the area of hand-raked racks is frequently obstructed in practical operation where the amount of debris in the water is considerable. This would increase the loss in the above example from 0.13 to 0.29 ft.

51. Loss of Head at Entrance (h_e). a. The basic equation is:

$$h_e = K_e \frac{V^2}{2g} \qquad (4\text{-}14)$$

where (V) is the velocity within the pipe at the entrance.

b. A direct relation between the coefficient of discharge for short tubes and the coefficient of eddy loss, K_e, is:

$$K_e = \frac{1}{C^2} - 1 \qquad (4\text{-}15)$$

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c. The loss of head in short tubes where there is no residual contraction of the jet (as in Figs. d, f, g, and h, Plate 202 and in the sluices used in Stewart's experiments given on Plate 203) may be assumed to be the same for similar entrances to closed conduits. The values of K_e derived from the experimental values of C_q of Chapter II are given below. Values of K_e may also be taken from Plate 801, Reservoir Outlet Conduits.

d. Coefficients of Eddy Loss for Eq. 4-14 /3/:

Fig.	Plate	Type	Coefficient K_e
g	202	Short tube with sharp-cornered entrance	0.56
d	202	Short tube with rounded entrance	0.24
f	202	Inwardly projecting tube with sharp-cornered entrance*	0.56 to 0.93
h	202	Inclined tube with sharp-cornered entrance	
		$\theta = 90^\circ$	0.49
		$\theta = 80^\circ$	0.56
		$\theta = 70^\circ$	0.65
		$\theta = 60^\circ$	0.73
		$\theta = 50^\circ$	0.78
		$\theta = 40^\circ$	0.88
		$\theta = 30^\circ$	0.93

* Depending on distance of projection

52. Loss of Head Due to Expansion of Pipe Section (h_{se}) and (h_{ge}).

a. For sudden expansion:

$$h_{se} = K_{se} \frac{V^2}{2g} \qquad (4\text{-}16)$$

where

$$K_{se} = \left[1 - \frac{A_1}{A_2}\right]^2 \qquad (4\text{-}17)$$

where A_1 and A_2 are the areas of the smaller and larger sections, respectively. The value of (V) for use in Eq. 4-16 is that in the smaller section. /3/ For sudden enlargements in open conduits and for the junction of a closed and open conduit:

$$K_{se} = \left[1 - \frac{A_1{}^2}{A_2{}^2}\right] C$$

where:

For square enlargements	C = 0.75
For enlargements with each side set at a 30-degree angle with the flume axis	C = 0.50
For perfectly designed enlargement transitions	C = 0.25

the value of (V) for use in Eq. 4-16 is that in the smaller section.

b. For gradual expansion:

$$h_{ge} = K_{ge} \frac{V^2}{2g} \qquad (4\text{-}19)$$

$$K_{ge} = \left(1 - \frac{A_1}{A_2}\right)^2 \sin\theta \qquad (4\text{-}20)$$

where (θ) is the angle between the axis of the pipe and the surface of the pipe. The value of (V) for use in Eq. 4-19 is that in the smaller section. /3/

53. Loss of Head Due to Any Obstruction in Pipes. The most common obstructions in pipes are valves partially open. The following formula, however, should apply approximately to any obstruction:

$$h_v = K_v \frac{V^2}{2g} \qquad (4\text{-}21)$$

where V = the velocity in the pipe in ft./sec. Values for K_v for ratios of the area of the opening in the obstruction to the area of the pipe are plotted on Plate 403. /1/ Loss of head through valves is discussed further in Chapter IX.

54. Losses Due to Contraction of Pipe Section (h_{sc}). a. For sudden contraction:

$$h_{sc} = K_{sc} \frac{V^2}{2g} \qquad (4\text{-}22)$$

where (V) is the velocity in the smaller pipe. Values of K_{sc} may be taken from Figure 3, Plate 401. /3/ Losses due to gradual contraction are negligible.

55. Loss of Head Due to Bends (h_b). a. Relatively Small Conduit:

$$h_b = K_b \frac{V^2}{2g} \qquad (4\text{-}23)$$

Values of K_b may be taken from Fig. 1, Plate 404, for 90° bends. Values of K_b for other than 90° bends may be obtained by finding the K_b for a 90° bend from Fig. 1, Plate 404, and multiplying by the correction factor obtained from Fig. 2, Plate 404. Fifty percent should be added to the values of K_b from Fig. 1 for screwed pipe elbows on account of the sudden enlargement and contraction in such fittings. For open conduits, use one-half the computed value of K_b. The velocity (V) is the mean velocity head in the bend. Another formula has been recomended /4/ in which K_b varies uniformly with depth from 0.125 (/90°)$^{0.5}$ at zero depth to 0.25 (/90°)$^{0.5}$ at full depth.   is the deflection angle of the bend in degrees. Values of K_b for use in Eq. 4-23 give the loss in excess of that which would occur in a straight pipe of equal length.

b. For a relatively large conduits and tunnels, see Par. 167, Chapter VIII, Reservoir Outlet Conduits.

56. Loss of Head Due to Fittings (h_y). a. The basic equation is:

$$h_y = K_y \frac{V^2}{2g} \qquad (4\text{-}24)$$

b. Recommended /3/ values of K_y for miscellaneous fittings are given in Fig. 3, Plate 404. These values are for all the flow in one direction. For uniting or dividing flow in such fittings, Ky may be obtained from Plate 405 /5/.

The notation used is indicated on Fig. 1, Plate 405, for the case of separation of the main stream and on Fig. 2, Plate 405, for the unification of the stream of a branch with that of a straight run. The diameter of the straight run of pipe, D, is constant throughout and the diameter of the branch is denoted by D_y. The following notations apply to Figs. 1 and 2, Plate 405:

- Q = total discharge in straight run of pipe
- D = diameter of straight run of pipe
- V = velocity corresponding to Q in a straight run of pipe
- Q_y = discharge carried by branch pipe
- V_y = velocity corresponding to Q_y in branch pipe
- D_y = diameter of branch pipe
- $h_y = K_y \frac{V^2}{2g}$ = loss of head at entrance to branch pipe in case of divergent flow, or loss of head in straight run just beyond junction of branch pipe in case of convergent flow.
- K_y = coefficient determined by experiment for divergent flow or for convergent flow.
- θ = angle of divergence or convergence in degrees.

c. Values in the tables are for pipes with sharp-edged junctions and for pipes with rounded junctions where the radius of rounding was 0.1 D_y.

57. Loss of Head Due to Exit (h_o). a. Loss of head at the outlet of a pipe is a special case of enlargement where the velocity in the second pipe is zero, or

$$\text{Lost head } (h_o) = \frac{(V-0)^2}{2g} = \frac{V^2}{2g} \qquad (4\text{-}25)$$

b. The loss of head coefficient is unity.

SECTION D: PIPE SYSTEMS

58. Simple Flow. a. Three types of problems are encountered in the computation of simple pipe flow; they may be outlined as follows: /6/

	Given	Required
(1)	D,L,Q or V, and n or f	h_f
(2)	h_f,D, L, and n or f	Q or V
(3)	h_f, Q, L, and n or f	D

b. For the first type, where the head loss is required, substitution in Eq. 4-6 yields the head loss.

c. For the second type, where the velocity or discharge is required, compute V by Eq. 4-6 and Q = AV.

d. For the third type, where the diameter is required, the following solution gives approximate pipe sizes. Through continuity, V is related to D and Q:

$$V = \frac{4Q}{\pi D^2} \qquad (4\text{-}26)$$

substituting in Eq. 4-6:

$$D^5 = \frac{8fL\,Q^2}{h_f g\ \ 2} \qquad (4\text{-}27)$$

or substituting:

$$f = \frac{184.8\ n^2}{D^{1/3}}$$

$$D^{16/3} = 4.65\ \frac{LQ^2\ n^2}{h_f} \qquad (4\text{-}28)$$

e. Approximate pipe sizes and solutions to simple flow problems can also be taken directly from the diagrams on Plates 406 and 407.

59. Series Pipes. a. When two or more pipes of different types and sizes are connected together in series, the discharge for steady flow conditions is the same in each pipe. The head loss for a series pipe system is the sum of the head losses for each individual pipe.

b. Two types of problems are encountered in series pipe systems: one in which the head loss is required for a given discharge and the other in which the discharge is required for a given head.

(1) Determining Head Loss for a Given Discharge. When the discharge is known in a series pipe system, the velocities and the velocity heads are immediately obtainable by the law of continuity. The various form losses and friction losses are expressed in sections B & C as a function of the velocity head. Therefore, the total head loss would be determined as the sum of all the losses due to friction and the various fittings. The total head loss would be expressed as follows:

$$H_L = (K_e+\ldots+K_f)\ V_1^2/2g + (K_f+\ldots+K_{sc})\ V_2^2/2g + \ldots + (K_f+\ldots+K_{se})\ V_o^2/2g \qquad (\text{Eq. } 4\text{-}29)$$

where K_e, K_f, K_{sc}, etc., are the various loss coefficients for each size of pipe. V_1, V_2, etc., are the velocities in each section of pipe in the series system. It should be noted that equation 4-29 expresses the total head loss in the system and not the total head.

(2) Determining Discharge for a Given Head. When the total head is known for a series pipe system, the discharge would be determined by writing the Bernoulli equation between the inlet water surface and the outlet including all the intervening losses. The total head would be equal to the sum of the total head loss determined by equation 4-29 and the outlet velocity head. The various form and friction loss coefficients would be determined from the geometry of the pipe systems, but the velocities would not be known for each section of the system. The average velocity in a pipe for conditions of steady flow is inversely proportional to the cross-sectional area. Therefore, the velocity heads in the Bernoulli equation would be expressed in terms of the outlet velocity head as follows:

$$V_1^2/2g = (A_o/A_1)^2\ V_o^2/2g = (D_o/D_1)^4\ V_o^2/2g \qquad (4\text{-}36)$$

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where

V_1 = the average velocity in any section of the pipe system
V_o = the average velocity in the outlet section
A_1 = the cross-sectional area of any section of pipe corresponding to V_1
A_o = the cross-sectional area of the outlet pipe section corresponding to V_o
D_1 = the diameter of any section of pipe corresponding to V_1
D_o = the diameter of the outlet pipe section

Expressing each velocity head in terms of the outlet velocity head, by equation 4-30, the total head would be determined from equation 4-29 as follows:

$$H = H_L + V_o^2/2g \qquad (4\text{-}31)$$

where

H = the total head of the system
$V_o^2/2g$ = the outlet velocity head
H_L = the total head loss of the system

c. Equivalent Lengths. Series problems may also be solved by using the principle of equivalent lengths (see Plate 409). The form losses, including entrance and exit, are expressed as equivalent lengths of one size pipe. Equating the Darcy-Weisbach formula to the form-loss expression

$$K \frac{V^2}{2g} = f \frac{L_e}{D} \frac{V^2}{2g} \qquad (4\text{-}32)$$

from which

$$\frac{L_e}{D} = \frac{K}{F} \qquad (4\text{-}33)$$

where L_e is the equivalent length of straight pipe of diameter D and resistance coefficient "f" for which $h_f = h_L$. The equivalent length in diameters is thus equal to the ratio K/f. Where f is known within reasonable limits, the form losses can be expressed as equivalent length and added to the actual length of conduit in order to simplify the calculations. By using the same principle, a pipe of given length, diameter, form, and roughness may be said to be equivalent to a different pipe if the overall head losses for the two pipes are the same for a given discharge.

60. Parallel Pipes. a. It is standard pipe line practice to lay a line parallel and connected to an existing line to increase the flow capacity. Two types of problems are encountered /6/:

(1) that of determining the capacity of the system when the elevation of the hydraulic grade line is known at the junction points.

(2) that of determining the division of a given flow among the pipes and the corresponding drop in grade line.

b. In parallel pipe problems (see Fig. 2, Plate 408), the head loss in all pipes are equal ($h_{f1} = h_{f2} = h_{f3}$), and the discharge through the parallel pipes must equal the discharge in the entrance and exit lines ($Q = Q_1 + Q_2 + Q_3$).

c. For the first type of problem (Plate 411), solve for the discharge in each pipe corresponding to the given drop in head and then add the discharges.

d. For the second type (Plate 412), proceed as follows:

(1) Assume a discharge, Q_1', in one pipe and compute the corresponding drop in head.

(2) Using this drop, compute the discharges Q_2', Q_3',..., in the other pipes.

(3) Apportion the given total discharge Q among the pipes in the same ratio as the computed trial discharges; that is,

$$Q_1 = \frac{Q_1' \times Q}{Q_1' + Q_2' + Q_3' + \ldots} \qquad Q_2 = \frac{Q_2' \times Q}{Q_1' + Q_2' + Q_3' + \ldots}$$

(4) Using the values Q_1, Q_2, Q_3, ..., obtained in this manner, compute the head loss along each pipe. If the results are not within the desired degree of accuracy, repeat the procedure using one of the new discharges.

61. Branching Pipes. a. The example typified by the "three-reservoir" problem of Fig. 3, Plate 408, is solved advantageously by application of hydraulic grade line principles /6/. The problem is to find the rate of flow through each pipe for particular elevations of the reservoir when lengths, diameters, and type of pipe are known. The hydraulic grade lines coincide with the water surface in each reservoir, and the slopes of these lines and the flow conditions are determined by the magnitude of the pressure head at the junction. The continuity equation may take either of the two forms:

$$Q_1 = Q_2 + Q_3 \qquad \text{or} \qquad Q_1 + Q_2 = Q_3$$

depending on whether the elevation of the hydraulic grade line at the junction point is greater or less than the elevation of B. The computation of the discharge for the example given in Fig. 3, Plate 408, and Plate 413, would be as follows:

(1) Assume the elevation of the hydraulic grade line at J (see Plate 408).

(2) Compute Q_1, Q_2, and Q_3 for this assumed condition

(3) If continuity requirements are satisfied, the problem is solved

(4) If the flow into the junction does not equal the flow out of the junction, make a new assumption of the grade-line elevation at the junction, raising the grade line at the junction point for greater flow in than out and lowering the grade line for less flow in than out

(5) Repeat the foregoing procedure until the continuity equation is satisfied

b. The same procedure is followed with more than three reservoirs. If they do not have a common junction, the elevation of the hydraulic grade line may be assumed at one junction point, and the corresponding discharges from the nearest two reservoirs may be computed. With these values, the elevation of the grade line may be computed at the second junction. If the continuity conditions at the second junction are not satisfied, a new assumption at the first junction must be made and the process repeated.

62. Examples. Several examples are given illustrating the method of computing the discharge and heads for various pipe systems.

a. Plate 408 depicts three pipe systems that are used as examples in later plates.

b. Plate 409 gives an example of two pipes in series connecting two reservoirs. The discharge is determined for a given total head by three methods: (1) solution of Bernoulli's Equation, (2) solution by equivalent lengths, and (3) solution by diagram.

c. Plate 410 shows the method of computing the discharge for a given head through a series pipe system with various fittings and pipe sizes.

d. Plate 411 gives an example of the method of determining the discharge through a system of three pipes connected in parallel for a given head loss. The discharge is determined by two methods, (1) solution by the Darcy-Weisbach Equation, and (2) solution by diagram.

e. Plate 412 shows the method of determining the head loss for a given discharge through a parallel pipe system. The head loss was determined by the Darcy-Weisbach Equation and by diagram.

f. Plate 413 shows the method of determining the discharge from three reservoirs connected by a series of branching pipes of different sizes and lengths.

63. References.

/1/ King, Horace W. Handbook of Hydraulics. New York, McGraw-Hill Book Co., 1939.
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/3/ Creager, William P., and Joel D. Justin. Hydroelectric Handbook. New York, John Wiley & Sons, 1950.
/4/ Hinds, Julian. Memorandum to Designing Engineer. "Lost Head in Pipe Due to Curvature". Bureau of Reclamation, 1919.
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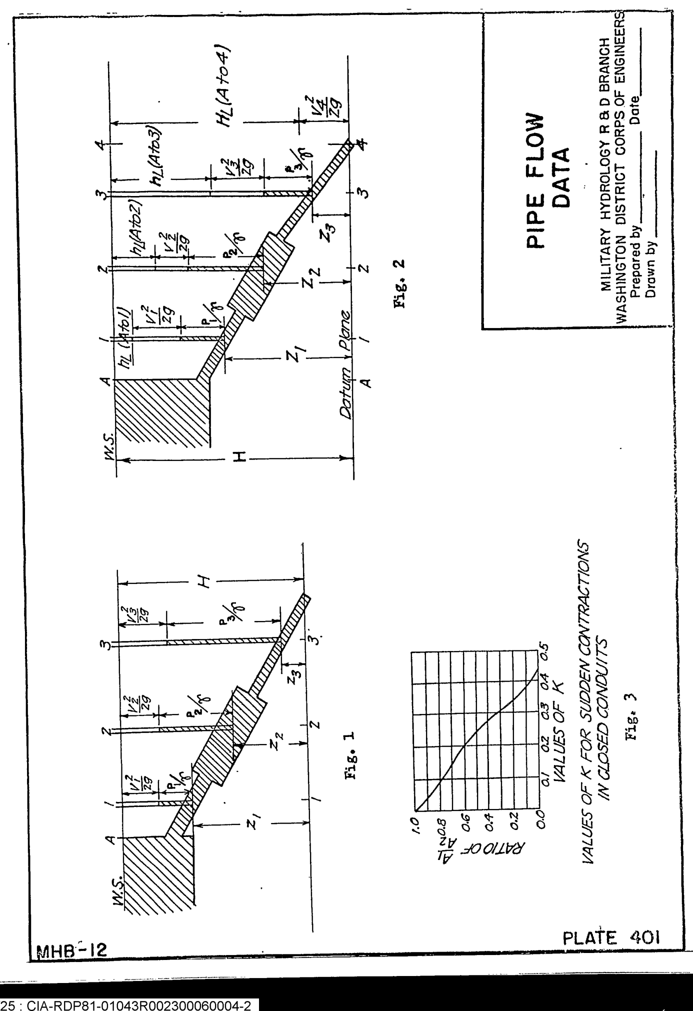

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Diameter of Pipe in inches	Mean Velocity (V) in Feet per Second								
	0.5	1.0	2.0	3.0	4.0	5.0	10.0	15.0	20.0
1/2	0.042	0.038	0.034	0.032	0.030	0.029	0.025	0.024	0.023
3/4	.041	.037	.033	.031	.029	.028	.025	.024	.023
1	.040	.035	.032	.030	.028	.027	.024	.023	.023
1 1/2	.038	.034	.031	.029	.028	.027	.024	.023	.023
2	.036	.033	.030	.028	.027	.026	.024	.023	.022
3	.035	.032	.029	.027	.026	.025	.023	.022	.022
4	.034	.031	.028	.026	.026	.025	.023	.022	.021
5	.033	.030	.027	.026	.025	.024	.022	.022	.021
6	.032	.029	.026	.025	.024	.024	.022	.021	.021
8	.030	.028	.025	.024	.023	.023	.021	.021	.020
10	.028	.026	.024	.023	.022	.022	.021	.020	.020
12	.027	.025	.023	.022	.022	.021	.020	.020	.019
14	.026	.024	.022	.022	.021	.021	.020	.019	.019
16	.024	.023	.022	.021	.020	.020	.019	.019	.018
18	.024	.022	.021	.020	.020	.020	.019	.018	.018
20	.023	.022	.020	.020	.019	.019	.018	.018	.018
24	.021	.020	.019	.019	.018	.018	.018	.017	.017
30	.019	.019	.018	.018	.017	.017	.017	.016	.016
36	.018	.017	.017	.016	.016	.016	.016	.015	.015
42	.016	.016	.016	.015	.015	.015	.015	.015	.014
48	.015	.015	.015	.015	.014	.014	.014	.014	.014
54	.014	.014	.014	.014	.014	.014	.013	.013	.013
60	.014	.013	.013	.013	.013	.013	.013	.013	.012
72	.013	.012	.012	.012	.012	.012	.012	.012	.012
84	.012	.012	.011	.011	.011	.011	.011	.011	.011

VALUES OF f IN THE DARCY-WEISBACK FORMULA, $h_f = f \frac{L}{D} \frac{V^2}{2g}$

For water flowing in straight smooth pipe

DARCY - WEISBACH COEFFICIENT

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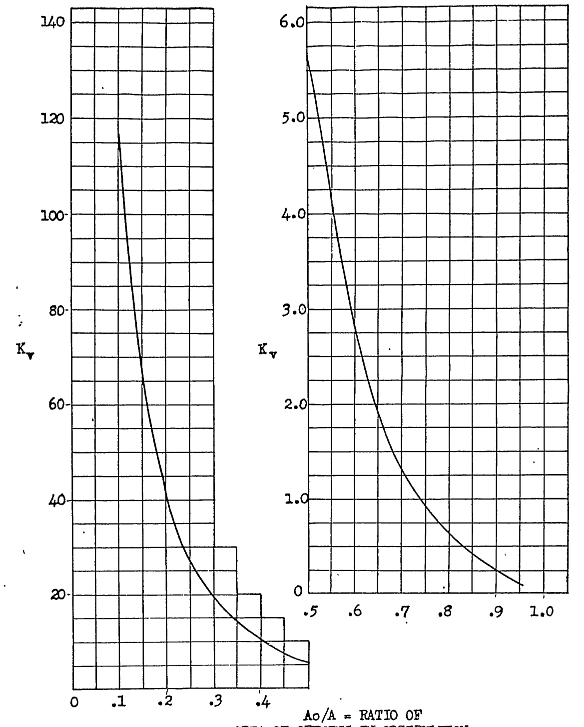

A_o/A = RATIO OF $\frac{\text{AREA OF OPENING IN OBSTRUCTION}}{\text{AREA OF PIPE}}$

VELOCITY HEAD COEFFICIENTS FOR OBSTRUCTIONS

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BEND LOSS COEFFICIENTS

VALUE OF K_b FOR VARIOUS DEGREE OF BENDS IN TERMS OF K_b FOR 90° BENDS
Fig. 2

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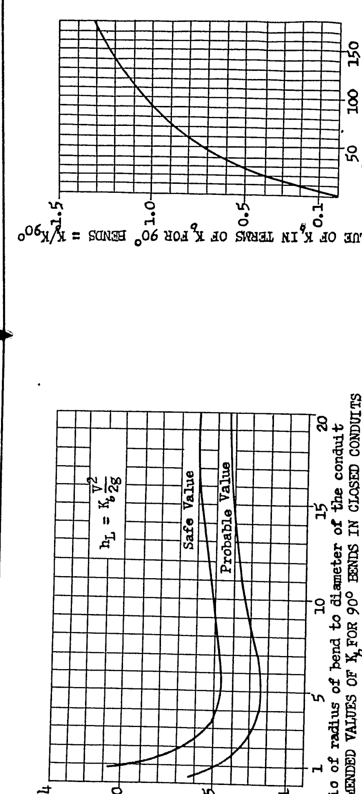
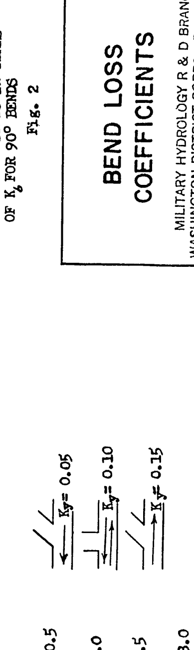


RECOMMENDED VALUES OF K_b FOR 90° BENDS IN CLOSED CONDUITS
Fig. 1



RECOMMENDED VALUES OF K_y FOR MISCELLANEOUS FITTINGS
Fig. 3

Angle of Divergence θ Deg.	$Q_y/Q = 0.3$		$Q_y/Q = 0.5$		$Q_y/Q = 0.7$	
	Sharp-edged	Rounded r=0.1 D_y	Sharp-edged	Rounded r=0.1 D_y	Sharp-edged	Rounded r=0.1 D_y
90	$D_y = D$ $V_y = 0.3V$ $K_y = 0.85$	$D_y = D$ $V_y = 0.3V$ $K_y = 0.76$	$D_y = D$ $V_y = 0.5V$ $K_y = 0.87$	$D_y = D$ $V_y = 0.7V$ $K_y = 0.74$	$D_y = D$ $V_y = 0.7V$ $K_y = 1.0$	$D_y = D$ $V_y = 0.7V$ $K_y = 0.8$
60	$D_y = D$ $V_y = 0.3V$ $K_y = 0.7$	$D_y = 0.61D$ $V_y = 0.8V$ $K_y = 0.59$	$D_y = D$ $V_y = 0.5V$ $K_y = 0.59$	$D_y = 0.79D$ $V_y = 0.8V$ $K_y = 0.54$	$D_y = D$ $V_y = 0.7V$ $K_y = 0.57$	$D_y = D$ $V_y = 0.7V$ $K_y = 0.52$
45	$D_y = 0.58D$ $V_y = 0.9V$ $K_y = 0.43$	$D_y = 0.58D$ $V_y = 0.9V$ $K_y = 0.35$	$D_y = D$ $V_y = 0.5V$ $K_y = 0.32$	$D_y = 0.75D$ $V_y = 0.9V$ $K_y = 0.32$	$D_y = D$ $V_y = 0.7V$ $K_y = 0.34$	$D_y = D$ $V_y = 0.7V$ $K_y = 0.3$

TABLE 1 - SEPARATION OF TWO STREAMS

Angle of Convergence Deg. θ	$Q_y/Q = 0.3$		$Q_y/Q = 0.5$	
	Sharp-edged	Rounded	Sharp-edged	Rounded
60	$D_y = 0.58D$ $V_y = 0.9V$ $K_y = 0.475$	$D_y = D$ $V_y = 0.3V$ $K_y = 0.33$	$D_y = 0.58D$ $V_y = 1.5V$ $K_y = 0.637$	$D_y = 0.58D$ $V_y = 1.5V$ $K_y = 0.563$
45	$D_y = 0.58D$ $V_y = 0.9V$ $K_y = 0.2$	$D_y = 0.58D$ $V_y = 0.9V$ $K_y = 0.2$	$D_y = 0.58D$ $V_y = 1.5V$ $K_y = 0.425$	$D_y = 0.58D$ $V_y = 1.5V$ $K_y = 0.425$
	$Q_y/Q = 0.7$		$Q_y/Q = 1.0$	
	Sharp-edged	Rounded	Sharp-edged	Rounded
60	$D_y = 0.58D$ $V_y = 2.0V$ $K_y = 0.715$	$D_y = 0.58D$ $V_y = 2.0V$ $K_y = 0.655$	$D_y = D$ $V_y = V$ $K_y = 0.645$	$D_y = D$ $V_y = V$ $K_y = 0.53$
45	$D_y = D$ $V_y = 0.7V$ $K_y = 0.54$	$D_y = D$ $V_y = 0.7V$ $K_y = 0.525$	$D_y = D$ $V_y = V$ $K_y = 0.38$	$D_y = D$ $V_y = V$ $K_y = 0.38$

TABLE 2 - UNIFICATION OF TWO STREAMS
($r = 0.1\ D_y$)

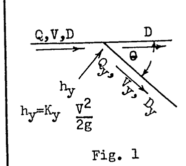


Fig. 1

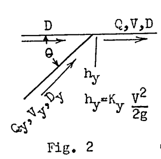


Fig. 2

DIVERGING & CONVERGING PIPE COEFFICIENTS

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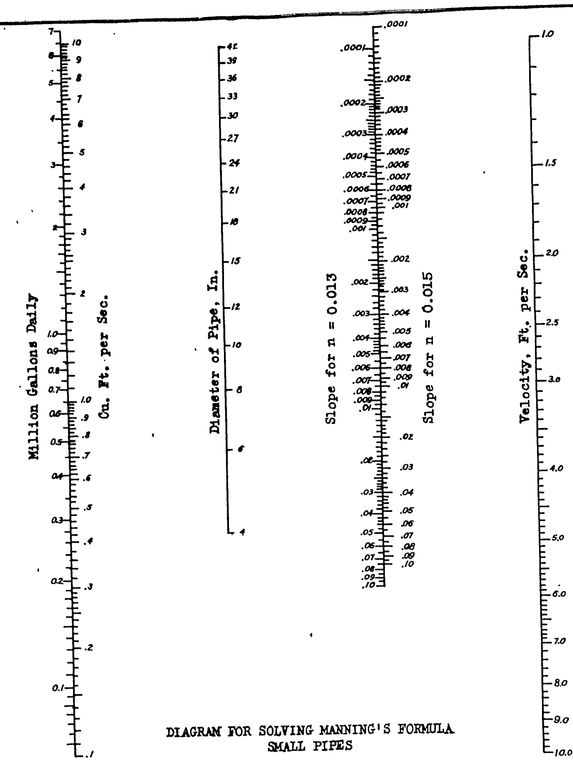

DIAGRAM FOR SOLVING MANNING'S FORMULA
SMALL PIPES

PIPE FLOW NOMOGRAPH

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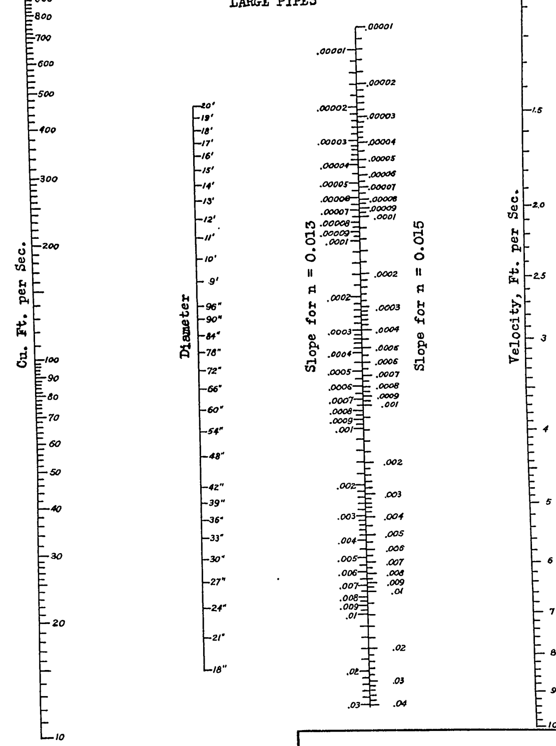
PLATE 406 MHB-12

DIAGRAM FOR SOLVING MANNING'S FORMULA
LARGE PIPES



PIPE FLOW NOMOGRAPH

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MHB-12 PLATE 407

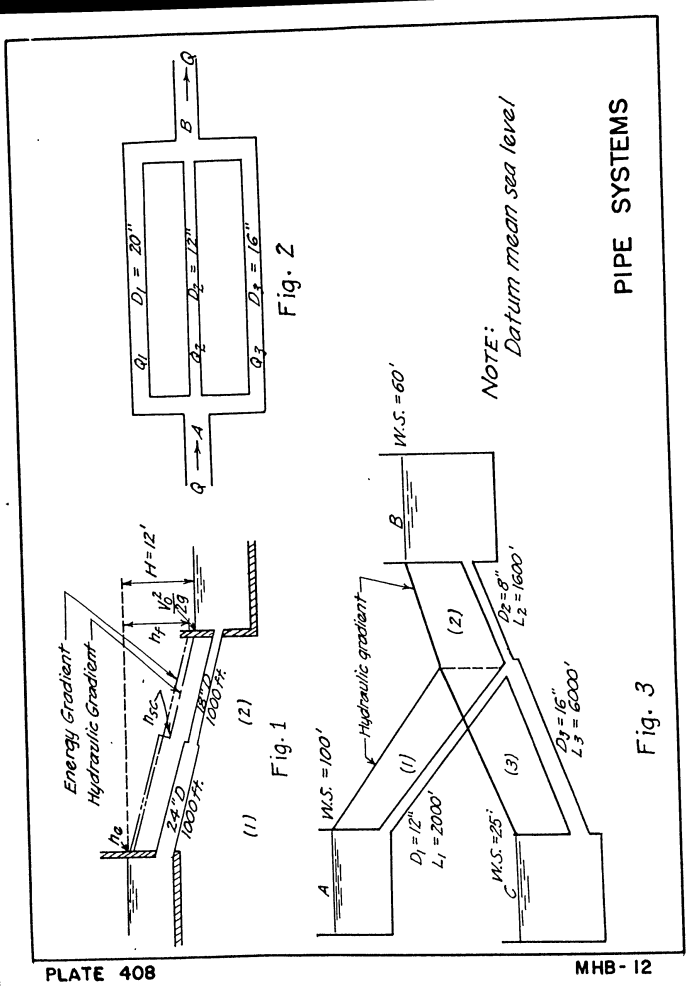

PLATE 408 MHB-12

DETERMINATION OF THE CAPACITY OF A SERIES PIPE SYSTEM

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(4) Assumed physical data for the pipe problem.

SOLUTION BY BERNOULLI EQUATION

(5) The Bernoulli equation was written from one reservoir surface to the other and included all the losses. H = 12', from Item 3; $K_e = 0.56$, from Par. 51; $f_1 = 0.025$ and $f_2 = 0.027$ were computed from equation 4-10; from Plate 401, $K_{sc} = 0.22$. The subscript 1 referred to the 24" pipe.

(6) The velocity head of the 24" pipe was expressed in terms of the 18" pipe by equation 4-36.

(7) Item (6) was substituted in item (5) and solved for V_2.

(8) The discharge was computed from:

$$Q = A_2 V_2 \qquad A = \frac{\quad}{4} (1.5)^2$$

SOLUTION BY EQUIVALENT LENGTHS

(9) All losses were expressed as equivalent lengths of 24" and 18" pipe by equation 4-33.

(10) The 18" line with form losses, equivalent in all to 1067.7 ft. of 18" pipe, was expressed as an equivalent length of 24" pipe -- that is, a length having the same head loss for the same discharge.

(11) The total length of 24" pipe and equivalent length of 24" pipe was determined as the sum of item (10) and the total equivalent length of 24" pipe of item (9).

(12) The velocity was computed by equation 4-6, using the computed equivalent length of 24" pipe of item (11).

(13) The discharge was computed from:

$$Q = A_1 V_1$$

SOLUTION BY DIAGRAM

(14) A discharge of 8 cfs was assumed. The friction slopes were determined from the diagram on Plate 407, knowing the discharge and the pipe diameters. The total length of equivalent pipe was computed as in item (9). The equivalent friction head loss was computed as the product of SL_e. The

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Total head was computed to be 7.34 ft. which was less than the 12 ft. head given in the problem. Two methods are available for determining the discharge. The first would be to assume a different discharge and repeat the above computations until the total head equalled 12 ft. The second method is described in the following items.

(15) The equivalent length of 24" diameter pipe that has the same loss as the 18" pipe was determined as follows:

Equivalent length of 24" pipe

$$= \frac{h_f \text{ of 18" pipe (Col. 5, Item 14)}}{\text{S of 24" pipe (Col. 3, Item 14)}}$$

$$= \frac{5.98}{0.0013}$$

LL = 4600 ft. of 24" pipe

(16) Total length of equivalent 24" pipe was determined as the sum of the lengths of item (15) and the total length of equivalent 24" pipe of item (9)

$$\text{Total } L_e = 4600 + 1045$$

$$= 5645 \text{ ft.}$$

(17) The slope for the total equivalent length of pipe of item (16) was determined from the total head of item (3) as follows:

$$S = \frac{H}{L_e} = \frac{12}{5645} = 0.00213$$

Enter the pipe diagram (Plate 406) with the 24" diameter and a slope of 0.00213 and read the discharge of 10.4 cfs.

When the pipe system is long, the actual pipe lengths should be used to simplify the computations instead of using the total equivalent lengths due to the form losses.

Item

(10) $$0.025 \frac{L_e}{2} \frac{V_1^2}{2g} = 0.027 \frac{1067.7}{1.5} \frac{V_2^2}{2g}$$

as

$$\frac{V_1^2}{2g} = 0.316 \frac{V_2^2}{2g}$$

$$L_e = \frac{2 \times 1067.7}{0.316 \times 1.5} \times \frac{0.027}{0.025} = 4865.5 \text{ ft. of 24" pipe}$$

(11) $4865.5 + 1000 + 44.8 = 5910.3$ ft. of 24" pipe

(12) $12 = 0.025 \frac{5910.3}{2} \frac{V_1^2}{2g}$ $V_1 = 3.24$ ft/sec

(13) $Q = \pi/4 \times 4 \times 3.24 = 10.2$ cfs

SOLUTION BY DIAGRAM

(14) First Trial

Q	Diameter	S	L_e	H = equiv. h_f
8	24"	0.0013	1044.8	1.36
8	18"	0.0056	1067.7	5.98
			Total head =	7.34 ft.

(15) The equivalent length of 24" pipe with the same losses as the 18" pipe $= \frac{5.98}{0.0013} = 4600$ ft.

(16) Total length of equivalent 24" pipe = 4600 + 1045 = 5645 ft.

(17) Enter pipe diagram with 24" diameter and new slope.

Diameter = 24"
Slope = 0.00213
Q = 10.4 cfs

SERIES PIPE
EXAMPLE

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DETERMINATION OF THE CAPACITY OF A SERIES PIPE SYSTEM

Item

INITIAL DATA

(1) Two concrete pressure pipes in series (Fig. 1, Plate 4403) 24" and 18" diameters.

(2) Length = 1000 ft. each

(3) Difference in reservoir surface levels = 12.0'

(4) Coefficient of roughness n = 0.013

SOLUTION BY BERNOULLI EQUATION

(5) $$H = K_e \frac{V_1^2}{2g} + f_1 \frac{L}{D_1} \frac{V_1^2}{2g} + K_{sc} \frac{V_2^2}{2g} + f_2 \frac{L}{D_2} \frac{V_2^2}{2g} + \frac{V_2^2}{2g}$$

$$12 = 0.56 \frac{V_1^2}{2g} + 0.025 \times \frac{1000}{2} \frac{V_1^2}{2g} + 0.22 \frac{V_2^2}{2g} + 0.027 \frac{1000}{1.5} \frac{V_2^2}{2g} + \frac{V_2^2}{2g}$$

(6) $$\frac{V_1^2}{2g} = (1.5/2)^4 \frac{V_2^2}{2g} = 0.316 \frac{V_2^2}{2g}$$

(7) $$12 = \frac{V_2^2}{2g} \left[0.56 \times 0.316 + (0.025 \times \frac{1000}{2}) \times 0.316 + 0.22 + 0.027 \times \frac{1000}{1.5} + 1\right]$$

$$12 = 23.347 \frac{V_2^2}{2g} \qquad V_2 = 5.7 \text{ ft/sec}$$

(8) $$Q = \frac{\pi}{4} \times (1.5)^2 \times 5.7 = 10.1 \text{ cfs}$$

SOLUTION BY EQUIVALENT LENGTHS

(9) For 24" Pipe

Entrance to 24" pipe $L_e = \frac{KD}{f} = \frac{0.56 \times 2}{0.025} = 44.8$

Equivalent length for loss in 24" pipe = 44.8

Total length of equivalent 24" pipe = 1044.8

For 18" Pipe

Contraction 18" pipe $L_e = \frac{KD}{f} = \frac{0.22 \times 1.5}{0.027} = 12.2$

Exit $L_e = \frac{1.0 \times 1.5}{0.027} = 55.5$

Equivalent length for loss in 18" pipe = 67.7

Total length of equivalent 18" pipe 1067.7

(10) $$0.025 \frac{L_e}{2} \frac{V_1^2}{2g} = 0.027 \frac{1067.7}{1.5} \frac{V_2^2}{2g}$$

as

$$\frac{V_1^2}{2g} = 0.316 \frac{V_2^2}{2g}$$

$$L_e = \frac{2 \times 1067.7}{0.316 \times 1.5} \times \frac{0.027}{0.025} = 4865.5 \text{ ft. of 24" pipe}$$

(11) 4865.5 + 1000 + 44.8 = 5910.3 ft. of 24" pipe

(12) $12 = 0.025 \frac{5910.3}{2} \frac{V_1^2}{2g}$ $\qquad V_1 = 3.24$ ft/sec

(13) $Q = \pi/4 \times 4 \times 3.24 = 10.2$ cfs

SOLUTION BY DIAGRAM

(14) First Trial

Q	Diameter	S	L_e	H = equiv. h_f
8	24"	0.0013	1044.8	1.36
8	18"	0.0056	1067.7	5.98
			Total head =	7.34 ft.

(15) The equivalent length of 24" pipe with the same losses as the 18" pipe $= \frac{5.98}{0.0013} = 4600$ ft.

(16) Total length of equivalent 24" pipe = 4600 + 1045 = 5645 ft.

(17) Enter pipe diagram with 24" diameter and new slope.

Diameter = 24"
Slope = 0.00213
Q = 10.4 cfs

SERIES PIPE EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ________ Date ________
Drawn by ________

DETERMINATION OF FLOW IN A COMPOSITE PIPE LINE

EXPLANATION OF COMPUTATIONS

INITIAL DATA

Item			
(1)	Assumed physical data of a composite pipe line.		

DISCHARGE COMPUTATIONS

The equations of the velocity head coefficients and the paragraph references are given below for each item of the computations. Each coefficient was expressed in terms of the outlet velocity head by use of equation 4-36.

Item			
(2)	Exit Velocity Loss	$K_o = 1.0$	(Par 57)
(3)	Screen Loss	$K_s = (1.45 - 0.45\,A_r - A_r^2)\,(A_o/A_s)^2$ A_r was assumed to equal 0.65	(Par. 50)
(4)	Entrance Loss	$K_e = 0.56\,(A_o/A_e)^2$	(Par. 51)
(5)	Gate Slot Loss	$K_g = 0.05\,(A_o/A_g)^2$	
(6)	Gradual Contraction	$K_{gc} = 0.01\,(A_o/A_{mean})^2$	(Par. 54)
(7)	Sudden Expansion	$K_{se} = (1 - A_1/A_2)^2\,(A_o/A_1)^2$ A_1 and A_2 were the areas of the smaller and the larger sections respectively	(Par. 52)
(8)	Bend Losses	$K_b = 0.25\,(\Delta/90^o)^{0.5}\,(A_o/A_b)^2$	(Par. 55)
(9)	Sudden Contraction	$K_{sc} =$ (Fig. 3, Plate 401, $A_1/A_2 = 12.57/28.27 = 0.445$)	(Par. 54)
(10)	Valve Loss	$K_v =$ (Plate 403)	(Par. 53)
(11)	Friction Losses	$K_f = \frac{C_f L}{R_m^{4/3}}\,(A_o/A_m)^2$ A_m = mean value of area for reach considered. R_m = mean hydraulic radius in feet for reach considered. L = Length, or equivalent length of pipe in feet. All areas are in square feet	(Par. 48)

(12) From equation 4-31, the total head of a pipe system is equal to the total head loss plus the outlet velocity head. The summation of the total head loss and the outlet velocity head was 5.416 or,

$$H = 4.416\,\frac{V_o^2}{2g} + \frac{V_o^2}{2g} = 5.416\,\frac{V_o^2}{2g}$$

The total head was initially given as 40 feet. Solving for the velocity and multiplying the velocity by the area of the outlet pipe, the discharge was found to be 273 cfs.

INITIAL DATA

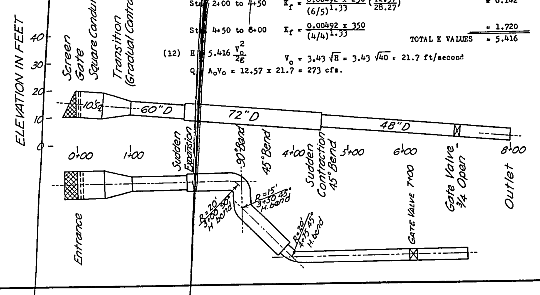
(1) A riveted steel pipe of varying diameter conveying water from a reservoir to discharge free at the lower end. Plan and profile are shown on Plate. (Manning's n = 0.013).

DISCHARGE COMPUTATIONS

Item			
(2)	Exit Velocity Loss	K_o	= 1.000
(3)	Screen Loss	$K_s = (1.45 - 0.45 \times 0.625 - 0.625^2)(\frac{12.57}{100.0})^2$	= 0.012
(4)	Entrance Loss Sta. 0+00	$K_e = 0.56\,(\frac{12.57}{100.0})^2$	= 0.009
(5)	Gate Slote Loss	$K_g = 0.05\,(\frac{12.57}{100.0})^2$	= 0.001
(6)	Gradual Contraction	$K_{gc} = 0.01\,(\frac{12.57}{100.0})^2$	= 0.000
(7)	Sudden Expansion Sta. 2+00	$K_{se} = (1.0 - \frac{19.63}{28.27})^2\,(\frac{12.57}{19.63})^2$	= 0.213
(8)	Bend Loss Sta. 3+00	$K_b = 0.25\sqrt{\frac{90}{90}}\,(\frac{12.57}{28.27})^2$	= 0.049
(8)	Bend Loss Sta. 3+50 Sta. 4+75	$K_b = 0.25\sqrt{\frac{45}{90}}\,(\frac{12.57}{28.27})^2$	= 0.035 = 0.175
(9)	Sudden Contraction Sta. 4+50	K_{sc}	= 0.250
(10)	Gate Valve 1/4 Closed Sta. 4+75	$K_v = 0.93\,(\frac{12.57}{9.45})^2$	= 1.650
(11)	Friction Losses Sta. 0+00 to 0+50	$K_f = \frac{0.00492 \times 50}{(\frac{100}{40})^{1.33}}\,(\frac{12.57}{100.0})^2$	= 0.001
	Sta. 0+50 to 1+00	$K_f = \frac{0.00492 \times 50}{(\frac{7.5}{4.0})^{1.33}}\,(\frac{12.57}{44.18})^2$	= 0.008
	Sta. 1+00 to 2+00	$K_f = \frac{0.00492 \times 100}{(5/4)^{1.33}}\,(\frac{12.57}{19.63})^2$	= 0.150
	Sta. 2+00 to 4+50	$K_f = \frac{0.00492 \times 250}{(6/5)^{1.33}}\,(\frac{12.57}{28.27})^2$	= 0.142
	Sta. 4+50 to 8+00	$K_f = \frac{0.00492 \times 350}{(4/4)^{1.33}}$	= 1.720
		TOTAL K VALUES	= 5.416

(12) $H = 5.416\,\frac{V_o^2}{2g}$ $\quad V_o = 3.43\sqrt{H} = 3.43\sqrt{40} = 21.7$ ft/second

$Q = A_oV_o = 12.57 \times 21.7 = 273$ cfs.



COMPOSITE PIPE EXAMPLE

DETERMINATION OF THE CAPACITY OF A PARALLEL PIPE SYSTEM
WHEN THE ELEVATIONS AT THE JUNCTION POINTS ARE KNOWN

Item INITIAL DATA

(1) Head loss A to B is 2.5 ft.

(2) Three concrete pressure pipes in parallel (Fig. 2, Plate 408), n = 0.013.

(3) $D_1 = 20''$ $L_1 = 2{,}500'$
$D_2 = 12''$ $L_2 = 800'$
$D_3 = 16''$ $L_3 = 1{,}500'$

SOLUTION BY DARCY - WEISBACH EQUATION

(4) $f_1 = \frac{184.8\,(0.013)^2}{(20/12)^{1/3}} = \frac{0.0312}{1.19} = 0.026$

$f_2 = 0.031$ $f_3 = 0.028$

(5) $2.5 = 0.026\,\frac{2500 \times V_1^2}{\frac{20}{12} \times 2g}$

$2.5 = 0.031\,\frac{800 \times V_2^2}{\frac{12}{12} \times 2g}$

$2.5 = 0.028\,\frac{1500 \times V_3^2}{\frac{16}{12} \times 2g}$

(6) $V_1 = 2.03$ ft/sec
$Q_1 = A_1V_1 = 2.18 \times 2.03 = 4.42$ cfs

$V_2 = 2.55$ ft/sec
$Q_2 = A_2V_2 = 0.785 \times 2.55 = 2.00$ cfs

$V_3 = 2.26$ ft/sec
$Q_3 = A_3V_3 = 1.40 \times 2.26 = \underline{3.16\text{ cfs}}$

(7) Total Capacity 9.58 cfs

SOLUTION BY DIAGRAM

(8) $S_1 = \frac{2.5}{2500} = 0.001$
$S_2 = \frac{2.5}{800} = 0.0031$
$S_3 = \frac{2.5}{1500} = 0.00167$

From Diagram (Plate 406)

$Q_1 = 4.40$ cfs
$Q_2 = 1.95$ cfs
$Q_3 = \underline{3.20\text{ cfs}}$
Q Total = 9.55 cfs

PARALLEL PIPE
EXAMPLE

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(3) Assumed physical data for the pipe problem.

SOLUTION BY DARCY - WEISBACH EQUATION

(4) The friction factor of each pipe was determined from equation 4-10.

$$f = \frac{184.8\,n^2}{D^{0.33}}$$

(5) Values were substituted in equation 4-6 and the velocity in each pipe was computed.

(6) Q_1, Q_2, And Q_3 were determined from Q = AV.

(7) Total capacity was determined as $Q_1 + Q_2 + Q_3 = 9.58$ cfs.

SOLUTION BY DIAGRAM

(8) The slope of the hydraulic gradient for each pipe was computed from the head loss of item (1) and the pipe lengths of item (3). Q was read for each slope and corresponding diameter from the diagram on Plate 406.

DETERMINATION OF THE DIVISION OF THE FLOW AND THE DROP IN GRADE
IN A PARALLEL PIPE SYSTEM

EXPLANATION OF COMPUTATIONS

INITIAL DATA

Item

(1) Assumed physical data for the pipe problem

COMPUTATION BY DARCY - WEISBACH FORMULA

(4) A reasonable value of Q_1 was assumed. The number of trial computations depended upon the validity of the assumed value.

(5) The drop in head hf_1 was computed by Eq. 4-6. f_1 was computed from Eq. 4-10 (see Item (4), Plate 411). $(Q_1/A_1)^2$ was substituted for V_1^2. Using the same head loss for h_{f_2} and h_{f_3}, Q_2' and Q_3' were computed.

(6) The given discharge Q = 9.60 cfs was apportioned among the pipes in the same ratio as the computed trial discharges, that is,

$$Q = \frac{Q_1' \times Q}{Q_1' + Q_2' + Q_3' + \ldots} \qquad Q = \frac{Q_2' \times Q}{Q_1' + Q_2' + Q_3' + \ldots}$$

Item

(7) Using the values Q_1, Q_2, Q_3, ..., the head loss along each pipe was computed. The results were sufficiently accurate and the division of flow was as computed in item (6).

SOLUTION BY DIAGRAM

(8) See item (4)

(9) From the diagram, Plate 406, for $D_1 = 20'$ and $Q_1' = 5.00$ cfs, $S_1 = 0.00135$ and h_L(A to B) = 0.00135 x 2500 = 3.38'. As the head losses in a parallel system are equal in each branch, S_2' and S_3' were computed. From the diagram, for $S_2' = 0.0042$ and $D_2 = 12'$, $Q_2' = 2.30$ cfs, for $S_3' = 0.00225$ and $D_3 = 16'$, $Q_3' = 3.70$ cfs. Total Q' = 11.00 cfs.

(10) Q was apportioned as in item (6)

(11) From the diagram, using values of Q_1, Q_2, Q_3 and corresponding values of D_1, D_2, D_3 values were read for S_1, S_2, S_3. The head loss was equal to S x L The values checked

COMPUTATIONS

Item

INITIAL DATA

(1) The flow Q at A and B (Fig. 2, Plate 408) is 9 60 cfs.

(2) Three concrete pressure pipes in parallel. n = 0.013

(3) $D_1 = 20''$ $L_1 = 2{,}500'$
$D_2 = 12''$ $L_2 = 800'$
$D_3 = 16''$ $L_3 = 1{,}500'$

SOLUTION BY DARCY-WEISBACH FORMULA

(4) $Q_1' = 5.0$ cfs

(5) $h_{f_1}' = 0.026\,\frac{2{,}500}{20/12}\,\frac{\left(\frac{5}{2.18}\right)^2}{2g} = 3.18'$

$3.18 = 0.031\,\frac{800}{12/12}\left(\frac{Q_2'}{.785}\right)^2/2g$

$3.18 = 0.028\,\frac{1{,}500}{16/12}\left(\frac{Q_3'}{1.40}\right)^2/2g$

$Q_1' = 5.0$ cfs
$Q_2' = 2.26$ cfs
$Q_3' = \underline{3.54\text{ cfs}}$
$Q'_{Total} = 10.80$ cfs

(6) $Q_1 = \frac{5.0 \times 9.6}{10.8} = 4.45$ cfs

$Q_2 = \frac{2.26 \times 9.6}{10.8} = 2.00$ cfs

$Q_3 = \frac{3.54 \times 9.6}{10.8} = \underline{3.14\text{ cfs}}$

9.59 cfs

(7) $h_{f_1} = 0.026\,\frac{2500}{20/12}\,\frac{\left(\frac{4.45}{2.18}\right)^2}{2g} = 2\ 52'$

$h_{f_2} = 0.031\,\frac{800}{12/12}\,\frac{\left(\frac{2.0}{.785}\right)^2}{2g} = 2.50'$

$h_{f_3} = 0.028\,\frac{1500}{16/12}\,\frac{\left(\frac{3.14}{1.40}\right)^2}{2g} = 2.48'$

SOLUTION BY DIAGRAM

(8) Assume $Q_1' = 5.0$ cfs

(9)

Diameter	Length	S'	h_L'	Q'
20"	2500	$S_1' = 0.00135$	0.00135 x 2500 = 3.38	$Q_1' = 5.00$ cfs
12"	800	$S_2' = \frac{3.38}{800} = 0.0042$	= 3.38	$Q_2' = 2.30$ cfs
16"	1500	$S_3' = \frac{3.38}{1500} = 0.0025$	= 3.38	$Q_3' = \underline{3.70\text{ cfs}}$
				$Q'_{Total} = 11.00$ cfs

(10) $Q_1 = \frac{5.0 \times 9.6}{11.0} = 4.36$ cfs

$Q_2 = \frac{2.30 \times 9.6}{11.0} = 2.00$ cfs

$Q_3 = \frac{3.70 \times 9.6}{11.0} = \underline{3.23\text{ cfs}}$

$Q_{Total} = 9.59$ cfs

(11) Check by diagram

S x L = 0.001 x 2500 - 2.50'
S x L = 0.0032 x 800 = 2.56'
S x L = 0.0017 x 1500= $\underline{2.55'}$

7.61'

PARALLEL PIPE
EXAMPLE

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DETERMINATION OF RATE OF FLOW IN BRANCHING PIPES

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(3) Assumed physical data for the pipe and reservoir system

COMPUTATIONS TO FIND Q_1, Q_2 and Q_3

(4) First Trial. The discharge through the pipe system was determined as described in the following steps:

1. The elevation of the hydraulic gradient at J was assumed to be 60 feet msl., in the first trial, thus neutralizing reservoir B and simplifying the computations.
2. The friction loss for L_1 and L_3 was determined as the difference in elevation between the reservoir water surface and the assumed hydraulic gradient at J.
3. The trial friction factor was assumed to be 0.020 for pipe line L_1 and L_3.
4. The velocity in L_1 and L_3 was computed by equation 4-6, to be 8.02 feet per second and 5.0 feet per second, respectively.
5. The assumed friction factors were checked for the computed velocities and pipe diameters by Plate 402.
6. The discharge was computed for each pipe line as the product of the velocity in step 4 and the cross sectional area of the pipe.
7. The discharge into the junction did not equal the discharge away from the junction, therefore, a new elevation of the hydraulic gradient was assumed.

(5) Second Trial. The elevation of the hydraulic gradient was assumed lower in the second trial to increase the discharge through pipes (1) and (2) and decrease the discharge in pipe (3). The computations were the same as in steps 1-7 above and the discharges were found to balance within 0.6% which was considered adequate.

DETERMINATION OF RATE OF FLOW IN BRANCHING PIPES

Item

INITIAL DATA

(1) Given the smooth pipe system shown in Figure 3, Plate 408.

(2)

		Reservoir W.S. elev.
L_1 = 2000'	D_1 = 12"	A = 100 ft. msl.
L_2 = 1600'	D_2 = 8"	B = 60 ft. msl.
L_3 = 6000'	D_3 = 16"	C = 25 ft. msl.

(3) Assume entrance and junction losses negligable.

COMPUTATIONS TO FIND Q_1, Q_2 and Q_3

(4) First Trial. Assume Elevation of hydraulic gradient at J = 60 ft. msl., trial friction factors f_1 and f_3 = 0.020.

$h_f = f_1 \frac{L_1}{D_1} \frac{V_1^2}{2g} = 0.020 \times 2000 \frac{V_1^2}{2g} = 40$ $\quad V_1 = 8.02$ ft. per second

$h_f = 0$ $\quad V_2 = 0$ ft. per second

$h_f = 0.020 \times \frac{6000}{1.33} \frac{V_3^2}{2g} = 35$ $\quad V_3 = 5.0$ ft. per second

$Q_1 = A_1 V_1 = 0.785 \times 8.02 = 6.3$ cfs

$Q_2 = 0$ cfs $\quad Q_1 + Q_2$ does not equal Q_3

$Q_3 = A_3 V_3 = 1.40 \times 5.0 = 6.9$ cfs

(5) Second Trial. Assume Elevation of hydraulic gradient at J = 58 ft. msl f_1 and f_3 = 0.020; f_2 = 0.027

$h_1 = 0.020 \times 2000 \frac{V_1^2}{2g} = 42$ $\quad h_2 = 0.027 \times \frac{1600}{.667} \frac{V_2^2}{2g} = 2$

$h_3 = 0.020 \times \frac{6000}{1.33} \frac{V_3^2}{2g} = 33$

$V_1 = 8.2$ ft/sec $\quad Q_1 = 6.33$ cfs

$V_2 = 1.41$ ft/sec $\quad Q_2 = \underline{0.50}$ cfs

$\quad\quad\quad\quad\quad\quad\quad\quad 6.83$ cfs

$V_3 = 4.86$ ft/sec $\quad Q_3 = 6.79$ cfs

Error = 100(6.83 - 6.79)/6.79 = 0.6%

BRANCHING PIPE EXAMPLE

CHAPTER V
OPEN CHANNELS

SECTION A: PRELIMINARY CONSIDERATIONS

64. Applications. The military hydrologist is concerned with problems in which the water flows with a free surface at atmospheric pressure as well as with problems of pressure flow in a closed conduit. Flow with a free water surface is termed open channel flow. Open channel flow problems include the computation of water surface profiles over spillways, through outlet conduits, or down chutes; the backwater effect of reservoirs or stillwater barriers; and the computation of open river rating curves.

65. Fundamentals. a. The prime difference between closed conduit flow as described in Chapter IV, and open channel flow, is the unique situation in which the boundary is no longer the prime governing factor in determining the pattern of flow. In problems of pipe flow, the pressure drop along the pipe in the direction of flow, depends on the energy losses and the condition at the ends of the pipe. Open channel flow, however, is characterized by the fact that the pressure /1/ is constant along the free surface and, therefore, the pressure variations within the fluid are basically determined by the principals of hydrostatics. The principals of hydrostatic pressure distribution does not hold for sharply divergent, convergent or curved flow as will be discussed later in this chapter.

b. Open channel flow may be laminar or turbulent, steady or unsteady, uniform or varied, tranquil or rapid. This chapter will not consider laminar flow as it has only minor application for military hydrology purposes. Also the problem of unsteady flow was not considered as it is covered in other bulletins of this series.

66. Scope. This chapter will explain the fundamental principals of open channel flow as applied to military hydrology. The second section of this chapter discusses the principals of uniform flow as determined by the Manning formula. The parameters of normal depth and critical depth, as well as the critical slope, are explained and illustrated by an example. The varied flow equation with its limitations is discussed in Section C. The twelve types of water surface profiles are classified and a procedure for profile analysis is discussed. The fourth section discusses the computation procedure to determine a backwater curve in a natural water course or a prismatic canal. The method of computing a tailwater rating curve in an open channel is also discussed in Section D. The hydraulic jump is discussed in Section E with the method of computing the jump in a rectangular and non-rectangular channel. Two examples were given: one showing the method of computing the M(y) function curve, and the other as a comprehensive example of determining the water surface profile in an outlet conduit with the hydraulic jump.

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SECTION B: UNIFORM FLOW

67. Definition. Steady uniform flow in open channels is a condition of flow in which the discharge is constant with respect to time and distance, along the channel. In steady uniform flow the resistance forces are exactly balanced by the force of gravity, and the water surface is parallel to the channel bottom. Steady uniform flow rarely, if ever, exists in natural channels; however, for practical purposes, prismatic channels of considerable length, constant cross section, straight alignment, and uniform slope can be considered to approach uniform flow. In treating varied flow, later in this chapter, uniform flow will be used as a reference.

68. Discharge Capacity. a. In analyzing the flow in an open channel, it is necessary to determine the discharges, velocities, slopes, roughness coefficients, depths, and water surface curves. The basic formula used in the analysis of channel flow, where both quantity and velocity are constant, is the Chezy Formula:

$$V = C(RS)^{0.5} \quad (5\text{-}1)$$

$$Q = AV = AC(RS)^{0.5} \quad (5\text{-}2)$$

where

V = mean velocity in feet per second
Q = discharge in cubic feet per second
C = an empirical coefficient determined from experiment
R = hydraulic radius in feet = A/P
A = area of cross section in square feet
P = The wetted perimeter of any channel or conduit, and is that portion of the perimeter of the cross section of the channel or conduit in contact with the liquid.
S = Friction slope, or energy gradient

b. Values of C for use in the Chezy Formula are usually computed by the Manning Formula:

$$C = \frac{1.486\, R^{1/6}}{n} \quad (5\text{-}3)$$

where

n = the coefficient of roughness of the channel

c. Equations 5-1 and 5-2 thus become:

$$V = \frac{1.486}{n}\, R^{2/3}S^{1/2} \quad (5\text{-}4)$$

$$Q = \frac{1.486}{n}\, AR^{2/3}S^{1/2} \quad (5\text{-}5)$$

69. Coefficient of Roughness. The concept of the coefficient of roughness as determined by modern fluid mechanics relates the friction factors in terms of the absolute roughness and the Reynolds Number. The Manning Formula with the Manning "n" has served as the basis of design for many years, and has proved suitable for the design of both open and closed conduits. The Manning "n", while empirical, has been used by the engineering profession to such an extent that it has been the standard for general use. A great deal of experimental data has been collected to establish the roughness coefficient for most conditions of flow. Plate 501 /2/ lists the values of the roughness coefficient for different materials and various flow conditions. Plates 502 and 503 /3/ consist of a chart and a nomograph for solving equation 5-4 in terms of the hydraulic radius, slope, and "n".

70. Conveyance. a. The discharge in an open channel as determined by equation 5-5 may be expressed as follows:

$$Q = \left[\frac{1.486}{n}\, AR^{2/3}\right] S^{1/2}$$

b. The terms in the brackets are a function of the boundary roughness, which is a constant for any specific cross section of a channel, and the depth of the cross section. This term /4/ is called the "conveyance" and designated as k'; therefore:

$$Q = k'\ S^{\,0.5} \quad (5\text{-}6)$$

where

$$k' = \frac{1.486}{n}\, AR^{2/3} \quad (5\text{-}7)$$

c. A curve termed the conveyance may be computed for any channel cross section if the roughness coefficient is known. The area and hydraulic radius are computed for various assumed depths and the corresponding value of k' found by use of equation 5-7. The values of the conveyance are plotted against the depths of flow. The smooth curve drawn through the plotted points is the conveyance curve.

71. Normal Depth. a. The depth of uniform flow constitutes a parameter that is fully determined for a given channel cross section, slope, channel roughness, and discharge. This depth is termed the "normal depth" and is designated as y_0. The normal depth of a given channel is determined as follows: Compute the conveyance curve for the cross section as described in paragraph 70. For a given discharge and bottom slope of the channel, substitute these values in equation 5-6 and solve for k'. Enter the conveyance curve with the value of k' computed in equation 5-6 and read the depth of flow. This depth of flow

is the normal depth for the specific discharge and the channel completely defined as to shape, roughness, and bottom slope.

b. In solving problems with varying discharges in a given canal, it is convenient to compute a normal discharge curve. This curve represents the discharges of uniform flow plotted against the normal depths for a fully defined canal. The normal discharge curve is computed as follows: Compute the conveyance curve of the cross section as described in paragraph 70. For various depths on the conveyance curve, determine the values of the conveyance. Compute the discharge by equation 5-6 from the values of the conveyance and the bottom slope. Plot the normal discharges against the normal depths and draw the normal discharge curve of uniform flow.

72. Critical Flow. a. Critical Depth. A second parameter of flow is termed the critical depth. The critical depth /4/ is that depth which has a minimum specific energy for a given discharge, and is designated as y_c. Since the critical depth is the depth at which the specific energy is a minimum, then it is the depth at which the first derivative of the specific energy equation is equal to zero, and is as follows:

$$\frac{A^3}{b_w} = \frac{Q^2}{g} \qquad (5\text{-}8)$$

where

A = cross-sectional area in sq. ft.
b_w = width of water surface in ft.
Q = discharge in cfs
g = acceleration of gravity

Equation 5-8 states that the critical depth for a given discharge in any cross section is equal to that depth at which the value of A^3/b_w is equal to Q^2/g. The left-hand term of equation 5-8 is a function only of the depth of flow in any given cross section. The term A^3/b_w will be designated $(m')^2$ and therefore:

$$m' = A(A/b_w)^{0.5} \qquad (5\text{-}9)$$

or

$$m' = Q/g^{0.5} \qquad (5\text{-}10)$$

The right-hand side of equation 5-9 will be termed the m' function. The m' function /4/ can be plotted for any canal cross section as a function of the depth "y". The critical depth is determined for any discharge in a canal as follows:

(1) Plot the value of each depth of a cross section against the value of m', as determined from equation 5-9.

(2) Determine the value of $Q/g^{0.5}$ for any assumed discharge.

(3) Enter the m' function curve determined in step 1, for the value of m' determined in step 2, and read the depth y. This depth is the critical depth for the given cross section and discharge. It should be noted that the critical depth for a given discharge is a function only of the channel cross section and is not affected by the channel slope or roughness.

b. Critical Discharge. Water flowing in a channel at or below the critical depth is described as being in a critical state. For a given discharge, the critical depth, as determined above, indicates the particular depth which makes the discharge flow in a critical state. Conversely, there is in a given channel for every depth a specific discharge that would flow as critical flow. This discharge is called the critical discharge /4/ and is designated as Q_c. Q_c is determined as follows:

$$Q_c = m'(g)^{0.5} \qquad (5\text{-}11)$$

A curve may be drawn with the critical discharge plotted against the depth for a given cross section. This curve would be the values of the product of 5.67 and the m' function plotted against the depth "y".

c. Critical Velocity. The velocity corresponding to the critical flow is called the critical velocity and is expressed as

$$V_c = \frac{Q_c}{A} = (gA/b_w)^{0.5} \qquad (5\text{-}12)$$

d. Critical Slope. The value of the bottom slope /4/ required to cause a given discharge to flow in a uniform state at the critical depth is termed the critical slope and is designated S_c. From the above definition and application of equations 5-6 and 5-10, the critical slope is computed as follows:

$$S_c = g\left[\frac{m'}{k'}\right]^2 \qquad (5\text{-}13)$$

Since m' and k' are a function of the depth in a given cross section, the critical slope can be plotted as a function of the depth.

e. Hydraulic Properties of Prismatic Cross Sections. Plates 504 and 505 show the hydraulic properties of a circular cross section in dimensionless terms as a function of the properties of a full section or of the diameter. The curves are of assistance in computing the k' and m' curves of any circular conduit. Plate 506 /5/ lists the equations of various hydraulic properties of several common prismatic channels.

73. Example. An example is given on Plate 507 showing the method of computation of the conveyance curve, the m' function curves, and the critical slope curve for a trapezoidal canal.

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SECTION C: VARIED FLOW

74. Definition. Varied flow in an open channel is a condition of flow in which the water surface is not parallel to the bottom. Varied flow consists of steady and unsteady non-uniform flow. Steady non-uniform flow is a condition of flow in which the discharge is constant at a section with respect to time, but the velocity varies with respect to distance along the channel. Unsteady non-uniform flow in open channels is a condition of flow in which the velocity and discharge is not constant with respect to time at a given section, nor with respect to distance along the channel. This section will be concerned only with steady non-uniform flow and will not consider the problem of unsteady flow. Gradually varied flow will be used in this chapter with the connotation of change with respect to distance and not with change with respect to time.

75. Equation of Varied Flow. a. The geometry of steady non-uniform flow is shown on Plate 102, and depicts the relationship between the channel bottom slope, the water surface slope, and the energy gradient or friction slope. In steady non-uniform flow /2/, the changing depth and velocity head would be expressed in terms of the bottom slope and friction slope as follows:

$$S_o - \frac{dy}{dx} = \frac{d}{dx}\left[\frac{v^2}{2g}\right] + S \qquad (5\text{-}14)$$

where

S_o = the channel bottom slope

S = the energy gradient or friction slope

dy/dx = the rate of change of the depth of flow with respect to distance along the channel

$\frac{d}{dx}\left[\frac{v^2}{2g}\right]$ = the rate of change of the velocity head with respect to distance along the channel

b. Equation 5-14 is the general differential equation of varied flow and is the basis for computing all water surface profiles. The rate of change of the velocity head /4/ is normally expressed in terms of the discharge, cross-sectional area and water surface width in the form

$$\frac{d}{dx}\left[\frac{v^2}{2g}\right] = -\frac{Q^2 b_w}{gA^3} \cdot \frac{dy}{dx} \qquad (5\text{-}15)$$

c. Substituting equation 5-15 in equation 5-16 and rearranging:

$$\frac{dy}{dx} = \frac{S_o - S}{1 - Q^2 b_w/gA^3} \qquad (5\text{-}17)$$

d. Equation 5-17 is the form of the differential equation of varied flow that is most commonly used for computation purposes. If the friction slope is expressed in terms of the discharge and conveyance by equation 5-6 and the value of the m' function is substituted in the denominator, equation 5-17 would be transformed as follows:

$$\frac{dy}{dx} = \frac{S_o - (Q/k')^2}{1 - \frac{1}{g}(Q/m')^2} \qquad (5\text{-}18)$$

e. If the changes in velocity head can be neglected, as is normal for large natural streams, equation 5-18 would be simplified to give:

$$\frac{dy}{dx} = S_o - (Q/k')^2 \qquad (5\text{-}19)$$

76. Limitations of Applicability of Varied Flow Equation. a. It is important to make clear the specific conditions under which equation 5-14 is applicable. As stated in paragraph 6, under any condition of fluid motion, if the effects of acceleration and tangential stresses are negligible, the flow /1/ occurs with hydrostatic pressure distribution. The distribution of pressure /4/ in a cross section of an open channel will obey the hydrostatic law, and will be affected solely by gravity, when and only when, flow takes place in such a manner that the fluid filaments have no acceleration components in the plane of the cross section. These conditions are satisfied when the stream lines have no appreciable divergence or curvature.

b. Divergent flow /4/ is defined as that condition of flow in which the stream lines are inclined to the channel cross section. If the angle of divergence is appreciable, the acceleration of the fluid filament will have components normal and parallel to the plane of the cross section. If the component of acceleration parallel to the plane of the cross section is appreciable it may not be neglected. Generally, the effect of divergence is comparatively small.

c. Curvilinear flow /4/ is defined as that condition of flow in which the stream lines are curved in the vertical plane. Flow that is curvilinear in a concave direction would cause a centrifugal force to act in the direction of gravity, and conversely, flow in a convex direction would cause the centrifugal force to act opposite to the direction of gravity.

d. A flow pattern that is curvilinear in a concave direction would therefore have a pressure distribution greater than hydrostatic by an amount equal to the centrifugal force caused by the curvilinear flow. The curvilinear flow pattern in a convex direction would have less than hydrostatic pressure distribution because of the centrifugal force acting opposite to the force of gravity. The deviations from hydrostatic pressure distribution caused by curvature are usually quite substantial so that when excessive curvilinear flow occurs, equation 5-14 is not strictly applicable.

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e. From the above discussion it should be noted that the varied flow equation is applicable only when the effects of curvature and divergence are sufficiently small to make the acceleration components negligible in the cross-sectional plane.

77. Water Surface Profiles. a. The water surface profile of a prismatic channel will change shape at each break in grade of the bottom slope or change in cross section. The rate of change of the water surface depth with respect to distance in a wide rectangular channel is a function of three dimensionless parameters: the bottom slope, the ratio of the normal depth to the actual depth, and the ratio of the critical depth to the actual depth. Various shapes of water surface profiles would be obtained by use of different combinations of the three parameters discussed above.

b. Surface Profile Classifications. The water surface, in a channel of prismatic cross section, may take any of twelve different shapes /6/ as shown on Plate 508. The twelve profiles are classified by a letter and a number. The letters are used to classify the bottom slope as to whether it is adverse (A), horizontal (H), less than the critical slope (M for mild), equal to the critical slope (C for critical), or greater than the critical slope (S for steep). The numbers refer to the location of the water surface profile with respect to the normal and critical depths, as follows: 1 for depths greater than either the normal or critical depth, 2 for depths between the normal and critical depth, and 3 for depths smaller than either the normal or critical depth.

c. Changes in Grade for Straight Prismatic Channels. /2/ Plate 509 shows the various water surface profiles that may occur at a break in grade for steady flow in a long prismatic channel. The first step in the analysis of the flow profiles is to compute the parameters of flow of the critical depth and normal depth for a given discharge. If the channel cross section remains the same, the critical depth remains the same for all slopes but the normal depth varies for each change in slope. Further description of individual cases follows:

(1) Figure (a) illustrates the change from a mild slope to a flatter mild slope. The normal depth is greater for the lesser mild slope, and both normal depths are greater than the critical. An M1 backwater curve forms above the mild slope at the left. It joins the greater normal depth above the break of grade. No curve may form over the lesser slope because (1) none of the backwater curves in channels of mild slope become tangent to the normal depth in the downstream direction, and (2) the flow is deeper than the critical throughout, so that the break in grade can affect the profile in the upstream direction only.

(2) Figure (b) shows a break in grade from a mild slope to a steeper mild slope. An M2 curve forms upstream from the change of grade, and joins the normal depth line of the steeper slope. This is also true for Fig. (c) where the change of grade is from a mild slope to a critical slope. Flow at or near the critical slope, however, is

especially subject to chance fluctuations, so that the profile in Fig. (c) cannot be predicted with as much assurance as that in Fig. (b).

(3) Figure (d) shows a transition from a mild slope to a steep slope. An M2 curve forms above the mild slope, and an S2 curve forms over the steep slope. The flow passes through the critical depth over the change of grade. According to the ordinary theory, both of the backwater curves become vertical as they approach the critical depth. Actually, the profile does not cross the critical depth vertically, because of the influence of the vertical components of acceleration which are assumed negligible in the theory of the varied flow equation. For the same reason, the critical depth may not occur precisely above the change of grade. If it is necessary to know the exact shape of the profile in the immediate neighborhood of the break in grade, recourse may be had to a model study. A short distance away from the break in grade, slopes become flat and the backwater curves apply with good accuracy.

(4) Figure (e) shows a change in grade from critical slope to mild slope. A C1 curve forms over the critical slope, connecting with the normal depth in the channel of mild slope. This represents a passing transition between the case of Fig. (a), in which the backwater curve extends a long distance upstream, and that of Fig. (g), in which the curve extends downstream (if the tailwater is shallow).

(5) Figure (f) shows the change from critical slope to steep slope. An S2 curve forms over the steep slope, starting from the critical depth over the brink. Here, again, the vertical components of velocity become of importance; the sharp corner called for by the backwater curve theory would be rounded, the effect extending a short distance up and down stream.

(6) Figure (g) shows a change of grade from steep to mild. Here different profiles may form depending upon the relative steepness of the two grades. If the normal depth on the mild slope is comparatively small, the swiftly flowing stream on the steep slope will continue flowing at uniform depth right up to the change of grade, where an M3 curve begins. This curve continues on down the mild slope, increasing in depth and decreasing in velocity until a hydraulic jump forms, after which flow continues at the normal depth on the mild slope. The location of the jump can be found by the method explained later in this paragraph. As the mild slope becomes flatter, the normal depth on it increases and the jump moves upstream. When the normal depth becomes comparatively large, the jump forms upstream from the change of grade, and is followed by an S1 curve joining the downstream normal depth over the change of grade.

(7) Figure (h) shows a change of grade from a steep to critical. Flow continues down the steep slope at its normal depth to the change of grade, where a C3 curve forms, extending horizontally across to its intersection with the normal depth on the critical slope. This is another transition case, the existence of which depends upon a delicate balance between the roughness and slope of the downstream portion of the channel.

(8) Figures (i) and (j) show the two possibilities when there is a break in grade with steep slopes on each side. The flow is normal down to the change of grade, after which an S3 curve or an S2 curve forms, depending upon whether the downstream grade is flatter or steeper than the upstream grade.

(9) Figures (k), (l), and (m) show changes from adverse slope to mild, critical, and steep slopes, respectively. An A2 curve forms over the adverse slope in each of them. This curve joins the normal depth line for the mild slope, but reaches the critical depth at the crest for the critical slope and the steep slope. An S2 curve forms on the steep slope, while above the critical slope the flow is at the normal depth from very near the crest. The sharp intersection of the normal depth line and the A2 curve would, of course, be rounded.

(10) A point of particular interest about the profiles shown in Figs. (k), (l), and (m), is that the discharge is not fixed by upstream channel conditions, as for all the other cases, but by the level of the horizontal asymptote of the A2 curve. Computation of the discharge, for a given elevation of the asymptote (or of the water level in a pool at the upstream end of the curve), is different for profile (k) than for profiles (l) and (m). Let us consider profiles (l) and (m) first. An approximate value of the discharge can be computed on the basis of assumptions that there is no loss of energy up to the crest, and that flow over the crest is at the critical depth. This approximate value of the discharge may be used to compute the value of the normal depth on the downstream slope, to check whether the slope is actually critical or steep, as assumed. (Until the discharge is known, it may not be certain whether the slope is mild or steep). Having tentatively determined that the flow is as shown in (l) or (m), and not (k), the A2 curve should be computed upstream to the source of supply at the upper end of the adverse slope. The pool level thus computed, which should include allowance for the velocity head at the entrance to the adverse slope, will be above the known pool elevation. If the difference is appreciable, a lower value should be assumed for the critical depth at the downstream end of the adverse slope, and the pool elevation recomputed. The computed pool elevations will lie on each side of the given pool elevation, if the second critical depth was chosen carefully, so that the discharge corresponding to the given pool elevation may be determined by interpolation.

(11) To determine the discharge when the profile is as shown in Fig. (k), first find the value of the discharge corresponding to the normal depth for which the total head at the crest is equal to the height of the pool level above the crest, and also the value corresponding to another slightly smaller normal depth. Then proceed as before, figuring back to the pool level and interpolating to find the correct discharge. Other methods of computation may be used for these cases, but the procedure outlined here is as simple as any, and has definite advantages when the discharge must be known for a range of pool elevations.

d. Analysis of Flow Profiles. The twelve water surface profiles on Plate 508 may be used to advantage in the preliminary determinations of the general shapes of the water-surface profiles in channels of uniform cross section when there are one or more changes in grade. If the discharge /6/ is known or assumed, the procedure is:

(1) Plot the grade line of the bottom of the channel to a greatly exaggerated scale.

(2) Compute the normal depth y_0 for each section of channel of constant grade, and plot a dashed line through the length of the section at a distance y_0 above the bottom.

(3) Compute the critical depth y_c and plot a dotted line at a distance y_c above the bottom.

(4) Locate all possible control points. These may occur (a) at the upstream end of any steep portion (if two or more adjacent sections are steep, the control point is at the upper end of the uppermost section); (b) at the upstream end of any long section of mild slope; (c) at the downstream end of any long section of steep slope; (d) at a weir, dam, or sluice gate.

(5) Starting at the critical depth over each control point determined in (4)(a) above, trace continuous water-surface profiles upstream above the critical depth and downstream below the critical depth, using only the appropriate portions of the twelve possible backwater curves. Trace similar profiles upstream from the normal depth over each control point of type (b) and downstream from each control point of type (c). Profiles may also have to be started from type (d) controls if there are any. The curves are sketched one by one, proceeding upstream if the depth is greater than critical, and downstream if it is less than critical.

(6) If there exist, in the partial profiles so plotted, flows at less than critical depth overlapped by flows at greater than critical depth, locate the hydraulic jump by finding where the depths sequent to the high-velocity flow intersects the profile of the tranquil flow. The procedure outlined above is based on the assumption that the discharge is known or can be determined from measurement.

e. Profile Analysis When the Total Head is Known. When the discharge is unknown, it cannot be stated for certain whether the actual slope is steep, mild, or critical. In the common case of flow from a lake or reservoir into the upper end of a channel, the discharge will be found to depend upon the total head and the flow profile. The procedure to be followed then is to assume a depth of flow at the nearest downstream point which might be a control, and to compute the water-surface profile upstream to the lake. If the value obtained does not agree with the known elevation of the lake surface, a different depth at the control is assumed, and the computation is repeated.

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SECTION D: BACKWATER CURVES

78. Applications. Backwater curves are required in predicting the water-surface profile to be expected in a given channel with a steady discharge. Several problems of the type occurring in military hydrology are the determination of the backwater effects of reservoirs and stillwater barriers, computation of open river rating curves, adjustments of rating curves for spillways that have long approach channels, and the determining of water surface profiles in chute spillways. Backwater curves are also applicable in computing the relationship between discharge and pool elevation in outlet conduits flowing partly full.

79. Water Surface Profiles. a. Water-surface profiles in channels can be determined most readily by use of Bernoulli's Theorem as described in Par. 12. The energy of the water at an upstream cross section is equal to the energy at the downstream section plus intervening energy losses. Applying Bernoulli's Theorem between Sections 1 and 2 in Fig. 2, Plate 102:

$$y_1 + Z + V_1^2/2g = y_2 + V_2^2/2g + h_f \qquad (5\text{-}20)$$

b. Rearranging and substituting terms gives the basic formula used in the backwater computation:

$$y_1 = y_2 + (V_2^2/2g - V_1^2/2g) - L(S_o - S) \qquad (5\text{-}21)$$

where

y_1 = upstream depth of water
y_2 = downstream depth of water
V_1 = average velocity in upstream cross section
V_2 = average velocity in downstream cross section
S_o = bottom slope
S = slope of energy gradient
L = length of reach

80. Friction Head Loss. a. The friction head loss, h_f in Equation 5-20 is computed by Manning's Equation:

$$Q = AV = \frac{1.486}{n} AR^{2/3}S^{1/2} \qquad (5\text{-}22)$$

b. Rearranging Equation 5-22, the formula for friction slope is:

$$S = \frac{V^2n^2}{2.21R^{4/3}} = \left[\frac{Q}{k'}\right]^2 = h_f/L \qquad (5\text{-}23)$$

where

k' = the conveyance as described in Par. 81.

c. For any reach the friction head "h_f" in Equation 5-20 is equal to the mean friction slope for the reach, multiplied by the reach length. The mean friction slope is normally computed as the mean of the friction slopes at the ends of the reach.

81. Cross Sections. a. In order to compute a backwater curve, data which determine the conveyance of the stream channel and overbank areas are necessary. Cross sections of the overbank may be plotted directly from an accurate topographic map. The shape of the stream may be obtained from hydrographic surveys or from soundings. Stations and cross sections are plotted on the thalweg of the stream.

b. Channel cross sections /7/ should be located where the cross-sectional areas begin to increase or decrease; where the roughness changes, or marked breaks in the bottom slope occur; and at regular intervals along reaches of uniform cross sections. On large rivers that have average slopes of 1 foot per mile or less, cross sections along fairly uniform reaches may be taken at intervals of a mile or more. In computing backwater curves on small tributaries that have very steep slopes it is frequently desirable to take cross sections at intervals of 1/4 mile or less. Cross sections of rivers at flood stages generally consist of two or more segments that have different values of the roughness coefficient "n". The main channel areas have a relatively low value of "n", and the one or more overbank areas, because of vegetation and artificial obstructions to flow, have a higher "n" value.

82. Reach Lengths. Although the ends of reaches are usually located at the cross sections referred to above, shorter reaches should be taken by estimating intermediate cross sections when the velocity change in a reach exceeds 20 percent. The accuracy of results is usually improved by use of relatively short reaches. Although reach lengths are usually measured along the stream thalweg, they should be measured along a line through the estimated center of mass of the flowing water when the line differs materially from the stream thalweg.

83. Selection of Manning's "n". The selection of the proper value of "n" is important. This may be accomplished by solving equation 5-22 for "n", when discharges corresponding to observed water surface profiles are known. Plate 501 gives values of "n" for average channel conditions, and is to be used as a guide when values of "n" cannot be determined from analysis of discharge records and profiles.

84. Starting Elevation. Backwater computations should be started at a point of control where the water surface elevation can be definitely determined. This may be at a gaging station, a dam, or a

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section where flow is at critical depth. If the starting elevation for the selected discharge cannot be determined readily the following procedure would be used: Estimate the maximum probable range of starting water-surface elevations, for a given discharge at a section downstream from the desired reach. A reasonable approximation may be computed by means of equation 5-22 to determine the uniform flow depth for the conditions of roughness and slope. Assume as a first trial starting elevation the estimated lower range limit, and compute a backwater curve to the section desired. Assume a second trial starting elevation at the upper limit of the estimated range and compute a backwater curve. The same starting location and the same discharge should be used for both computations. If the selected location is sufficiently far downstream and if the range of starting elevations are near the true elevation, the two backwater curves will merge into one before the computations have progressed to the desired reach. If the backwater curves do not merge, the starting location should be located farther downstream and the computations repeated.

85. Computation Procedure. a. The method of computing the water surface profile /7/ is based on the Manning Formula written in the following form for any cross section:

$$Q = k' \; (h_f/L)^{0.5} \qquad (5\text{-}24)$$

where

Q = discharge in the reach in cfs
k'= the conveyance of the mean cross section in the reach
h_f= the friction head loss in the reach in ft.
L = the reach length in ft.

For any particular cross-sectional area, the drop in the energy gradient of the reach is given by:

$$h_f = L\left[\frac{Q}{k'}\right]^2 \qquad (5\text{-}25)$$

When the channel cross section is divided into several component parts with different reach lengths and roughness coefficients, equation 5-25 for friction head loss becomes:

$$h_f = \left[\frac{L_1 + L_2 + \dots + L_n}{(k_1')^2 + (k_2')^2 + \dots + (k_n')^2}\right] Q^2 \qquad (5\text{-}26)$$

where: $L_1, L_2 \dots L_n$ = the length of reach corresponding to the component part of the total cross section represented by each subscript number

$k_1', k_2' \dots k_n'$ = the conveyance of the component part of the total cross section represented by each subscript number.

b. Profiles for Natural Watercourses. One method of computing the water surface profile for a natural watercourse is described below, and while not an exact method, is adequate for military hydrology purposes. This method is based on equation 5-24 and assumes that the velocity-head changes in each reach are negligible. Also it will be assumed that a mean cross section can be selected in the reach that represents the hydraulic properties of the entire reach. It is also tacitly assumed that the flow conditions are such that the limitations on the applicability of the varied flow equation as described in paragraph 76, are not exceeded. The method of computing the water surface profile of a natural watercourse would be as follows:

(1) Divide the stream into reaches of moderate lengths as described in paragraph 82.

(2) Select a representative cross section in each reach and determine its location in the reach.

(3) Assume a trial water surface elevation at the representative cross section of the first reach.

(4) The total fall of the water surface in the reach is computed by use of equation 5-25 and the hydraulic properties of the representative section at the assumed elevation.

(5) The total fall in the reach is then proportioned by the ratio of the reach length to the mean section length to determine the fall from the mean section to the lower end of the reach. This proportioned fall is then added to the starting water surface elevation.

(6) If the trial and computed elevations check within the desired accuracy, the total fall is then added to the starting water surface elevation to give the water surface elevation at the upper end of the reach.

(7) Steps (3) to (6) inclusive are repeated for each reach. the upper water surface elevation of the first reach becomes the lower water surface elevation of the second reach, and so on for the number of reaches considered.

(8) The water surface elevations at the ends of each reach are connected by a slightly curved line to give the water surface profile of the stream for the given discharge.

c. Profiles For Prismatic Channels. The general differential equation of varied flow as described in Paragraph 75 may be transformed into an equation of differentials in terms of the specific head and a finite channel length as follows:

$$\Delta L = \frac{\Delta H_o}{S_o - S} \qquad (5\text{-}27)$$

If the energy gradient is assumed to be the mean of the energy slopes at each end of a reach for uniform flow then equation 5-25 would be:

$$\Delta L = \frac{(V_2^2/2g + y_2) - (V_1^2/2g + y_1)}{S_o - (Q/k')_m^2} \qquad (5\text{-}28)$$

where: The subscript 1 indicates the velocity head and depth of flow at the upstream cross section of the reach.
The subscript 2 indicates the velocity head and depth of flow at the downstream cross section of the reach.
$(Q/k')_m^2$ is the average or mean energy slope computed from the hydraulic properties of the cross sections at each end of the reach.

In applying equation 5-28 the following rule should be kept in mind: when the depth of flow is greater than critical, the computations should be carried upstream and when the depth is less than critical, the computations should be carried downstream. The method of computing the water surface profile /8/ for a prismatic channel would be as follows:

(1) Compute the conveyance and m' function curves of the cross section as described in paragraphs 70 and 72, respectively.

(2) For each discharge considered, determine the normal depth and critical depth as described in paragraphs 71 and 72.

(3) From the known starting elevation and the values of y_o and y_c, determine whether the computation would proceed upstream or downstream.

(4) Assume a small change in water surface elevation.

(5) Compute the values of the energy slope by equation 5-23 for the given starting depth and the assumed depth of step (4).

(6) The specific energy is computed for the starting and upstream depths by equation 1304.

(7) Equation 5-26 is used to determine the length of reach that would be required to give the assumed difference in the depth of flow of step (4).

(8) The change in the water surface elevation of step (4) is plotted at the station corresponding to that computed in step (7).

(9) Steps (4) to (8) are repeated until the entire profile is determined.

86. Tailwater Rating Curve. A tailwater rating curve is a graph showing the relationship of the water-surface elevation immediately downstream from the dam for each discharge passing the structure. A tailwater rating curve is determined by computing a series of water surface profiles up the river for various discharges and plotting the discharge against the water surface elevation at the dam. The backwater computation should start from a known rating station below the dam or from a point that is a sufficient distance from the dam to eliminate any error in the assumed starting elevation as described in Par. 84. Assume a discharge and a reasonable starting elevation at a point downstream from the dam and compute the backwater to the dam by

the methods described in Par. 85. Repeat the above process for the estimated range of discharges from the dam and plot the discharges against the resulting water surface elevations.

87. Examples. a. The computation of the discharge in an irregular channel is shown on Plate 510.

b. Plate 511 shows the method of computing a backwater curve in a natural channel. The reach lengths, cross sections, channel roughness, and starting elevations were selected by the methods given in paragraphs 81 to 84, inclusive. Conveyance curves were plotted, but not shown, for each section, as a function of the water surface elevation.

SECTION E: HYDRAULIC JUMP

88. Definitions. a. The hydraulic jump is a local phenomenon by means of which flow passes in an abrupt manner from a rapid to tranquil state.

b. There are two distinct types of jumps: The shock-type and the undular-type jumps. The shock-type jump is distinguished by its turbulent and frothy roller which forms on the abrupt rise of the water surface. Observed in a glass-walled flume, the shock-type jump features an underlying expanding jet of water. The jet is covered by a surface roller in which the particles are engaged in a circuitous motion and do not move downstream. The undular-type jump is characteristic of jumps of comparatively low height. It is mostly observed in natural streams of moderately steep bottom slopes. The transition from the lower to the upper stage features a series of undulations or small waves of diminishing size.

c. In rapid flow the hydraulic jump may be caused by an obstacle or sill in the channel, a change in the channel cross section, a decrease in the bottom slope, or an abrupt rise in the channel bottom. In each of these cases the initial depths would be below the critical depth and the downstream or final depth would be above the critical depth.

89. The General Hydraulic Jump Equation. The hydraulic jump phenomenon is analysed by the use of the laws of fluid statics, impulse-momentum, and specific energy, and the results of such analysis closely agree with experimental observation. The decrease in the velocity of the fluid between the upstream and downstream sections of the hydraulic jump results in a decrease in the momentum of the fluid. The impulse momentum law as given in Par. 13, states that the decrease in momentum is caused by an unbalanced force continually acting on the fluid mass between the sections. The unbalanced force is the difference in the total hydrostatic force in upstream and downstream sections. From these relationships the basic general equation for the hydraulic jump /9/ in channels of any cross section would be as follows:

$$A_2\bar{y}_2 - A_1\bar{y}_1 = \frac{Q^2}{g}\left(\frac{1}{A_1} - \frac{1}{A_2}\right) \qquad (5\text{-}29)$$

where A_1 & A_2 = cross-sectional area above and below the hydraulic jump respectively
$\bar{y}_1$ & $\bar{y}_2$ = depth below the water surface of the centroid of sections A_1 & A_2 respectively
Q = discharge in cfs
g = acceleration of gravity

90. The Hydraulic Jump in a Rectangular Channel. a. The hydraulic jump in a rectangular channel is a function of the Froude number and a ratio of the initial and final depths of flow. In a rectangular channel equation 5-29 would be simplified by substitution of the proper relationship for area, centroid depth, and discharge in terms of width, depth and discharge per unit width. The equation for the hydraulic jump in a rectangular channel is as follows:

$$y_2 = \frac{y_1}{2}\left[-1 + \left(1 + \frac{8q^2}{gy_1^{\,3}}\right)^{0.5}\right] \qquad (5\text{-}30)$$

or

$$y_2/y_1 = 1/2\left[\left(1 + 8N_f^{\,2}\right)^{0.5} - 1\right] \qquad (5\text{-}31)$$

where
y_2 = depth of flow below the hydraulic jump
y_1 = depth of flow above the hydraulic jump
q = discharge per unit width of channel
g = acceleration of gravity
N_f = Froude number

b. Equation 5-30 gives a simple relationship between the three variables, initial and final depths of the hydraulic jump and the unit discharge. If any two of the three variables are known, the third variable can be determined. Equation 5-30 may be solved by use of the chart /10/, shown on Plate 512, which depicts the discharge per foot of width as a function of the initial and final depths of the hydraulic jump.

91. The Hydraulic Jump in a Non-Rectangular Channel. a. The relation between the initial and final depths of the hydraulic jump in a non-rectangular channel cannot be expressed by a simple equation as for rectangular sections. The initial and final depths of the hydraulic jump can be determined by a graphical method. Equation 5-29 may be transposed as follows:

$$\frac{Q^2}{gA_1} + A_1\bar{y}_1 = \frac{Q^2}{gA_2} + A_2\bar{y}_2 \qquad (5\text{-}32)$$

b. It is evident from equation 5-32 that the variables are a function of the depth of flow for a given discharge. Also the depths y_1 and y_2 correspond to two equal values of a certain function, to be designated:

$$M(y) = \frac{Q^2}{Ag} + A\bar{y}$$

c. Thus, for any given shape cross section and discharge, a curve can be obtained by solving equation 5-23 for each assumed increment of flow depth. The curve obtained from plotting M(y) against y possesses a minimum value of M(y) at the critical depth as shown on Plate 513. After construction of this curve and with one depth known, the other depth may be determined by passing a vertical line through the point of known depth and reading the new depth on the other branch of the curve, the downstream or final depth of the hydraulic jump is termed the sequent depth or conjugate depth of the upstream or initial depth of flow. Plate 513 shows a specific energy diagram superimposed on the M(y) diagram. An initial depth y_1 is shown on the M(y) and H_o curves at points 1 and 2, respectively. The "sequent depth" (y_2) is shown on the M(y) curve at point 3. The depth on the specific energy curve having the same specific energy as point 2 is shown as point 4 and is termed the "alternate depth". The alternate depth is larger than the sequent depth by an amount equal to the difference in elevation corresponding to the energy lost in the hydraulic jump. Therefore, the differences in the quantity of energy between points 3 and 4 represents the amount of energy dissipated in the form of heat and turbulence in the hydraulic jump.

92. Examples. a. The computation of the H_o and M(y) curves /4/ for a trapezoidal canal are shown on Plate 513.

b. Plate 514 shows a nomograph /11/ used in computing the hydraulic jump in a circular conduit flowing partially full. Several examples are given on the plate to illustrate the method of using the nomograph.

c. Plate 515 shows the method of computing the water surface profile /8/ in the outlet conduit of a large dam. This is a comprehensive example, illustrating the method of determining the water surface profiles of a prismatic channel as well as determining the height and location of the hydraulic jump.

93. References.

/1/ Rouse, H. Elementary Mechanics of Fluids. New York: John Wiley and Sons, 1946.

/2/ Woodward, S. M. and Posey, C. S. Hydraulics of Steady Flow in Open Channels. New York: John Wiley and Sons, 1941.

/3/ Mavis, F. T. The Construction of Nomographic Charts. Scranton: International Textbook Company, 1939.

/4/ Bakhmeteff, B. A. Hydraulics of Open Channels. New York: McGraw-Hill Book Co. Inc., 1932.

/5/ King, H. W. Handbook of Hydraulics. New York: McGraw-Hill Book Co. Inc., 1939.

/6/ Rouse, H. (Editor). Engineering Hydraulics. New York: John Wiley and Sons, 1950.

/7/ "Part CXIV, Hydrologic and Hydraulic Analyses, Chapter 9, Computation of Backwater Curves in River Channels." Engineering Manual for Civil Works. Office, Chief of Engineers, Corps of Engineers, Dept. of the Army.

/8/ Preliminary Draft "Part CXVI, Hydraulic Design, Chapter 2, Reservoir Outlet Structures". Engineering Manual for Civil Works. Office, Chief of Engineers, Corps of Engineers, Dept. of the Army.

/9/ Vennard, J. K. Elementary Fluid Mechanics. New York: John Wiley and Sons, 1940.

/10/ "Requirements for the Hydraulic Jump Below Spillways". Office of the Division Engineer, Ohio River Division, Corps of Engineers, Dept. of the Army. June 1941.

/11/ Mavis, F. T. "Critical Flow in Circular Conduits Analyzed by Nomograph". Engineering News Record, March 7, 1946.

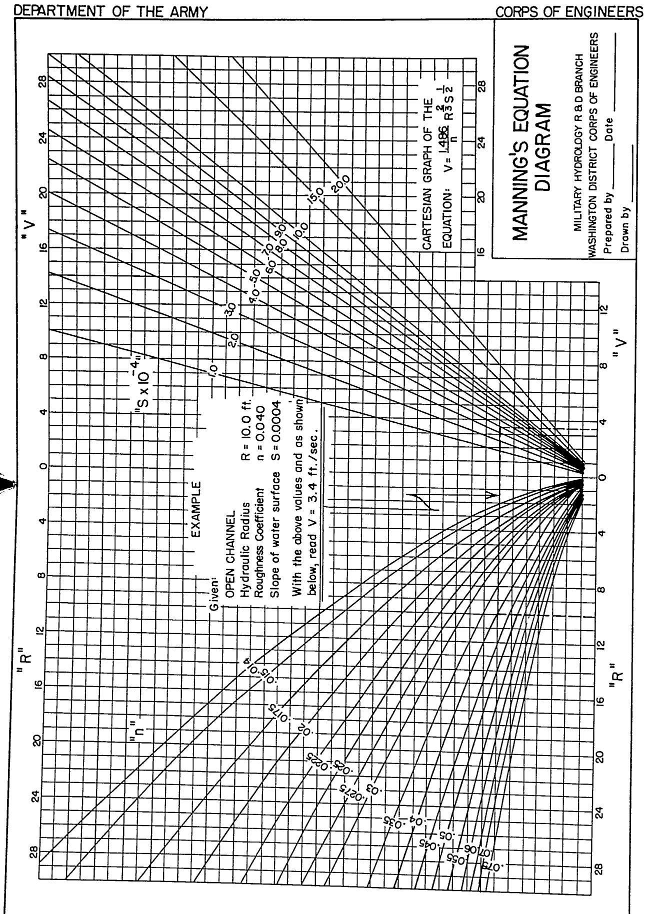
VALUES OF "n" FOR USE IN MANNING'S OR KUTTER'S FORMULAS

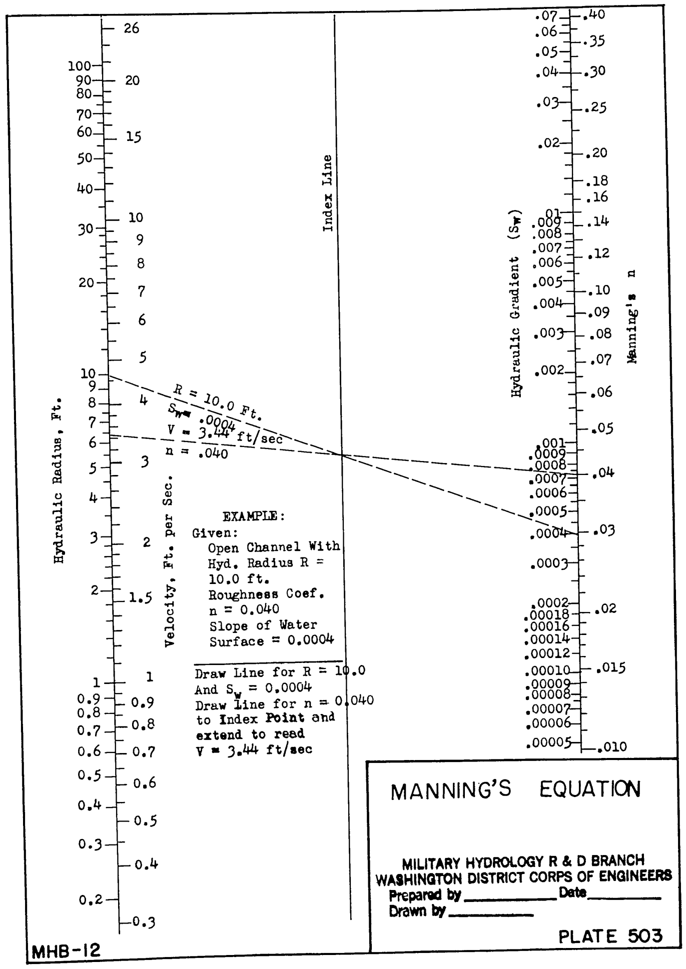
0.009 and 0.010	Very smooth and true surfaces, without projections. Clean new glass, pyralin, or brass, with straight alignment.
0.011 and 0.012	Smoothest clean wood, metal, or concrete surfaces, without projections, and with straight alignment.
0.013	Smooth wood, metal, or concrete surfaces without projections, free from algae or insect growth, and with reasonably straight alignment.
0.014	Good wood, metal, or concrete surfaces with very small projection, with some curvature, with slight insect or algae growth, or with slight gravel deposition. Shot concrete surfaced with troweled mortar.
0.015	Wood with algae and moss growth, concrete with smooth sides but roughly troweled or shot bottom, metal with shallow projections. Same with smoother surface but excessive curvature.
0.016	Metal flumes with large projections into the section. Wood or concrete with heavy algae, or moss growths.
0.017	Shot concrete, not troweled, but fairly uniform.
0.018-0.025	Metal flumes with large projections into the section and excessive curvature, growths, or accumulated debris.
0.016-0.017	Smoothest natural earth channels, free from growths, with straight alignment.
0.020	Smooth natural earth, free from growths, little curvature. Very large canals in good condition.
0.0225	Average, well-constructed, moderate-sized earth canal in good condition.
0.025	Very small earth canals or ditches in good condition, or larger canals with some growth on banks or scattered cobbles in bed.
0.030	Canals with considerable aquatic growth. Rock cuts, based on average actual section. Natural streams with good alignment, fairly constant section. Large floodway channels, well maintained.
0.035	Canals half choked with moss growth. Cleared but not continuously maintained floodways.
0.040-0.050	Mountain streams in clean loose cobbles. Rivers with variable section and some vegetation growing in banks. Canals with very heavy aquatic growths.
0.060-0.075	Rivers with fairly straight alignment and cross section, badly obstructed by small trees, very little underbrush or aquatic growth.
0.100	Rivers with irregular alignment and cross section, moderately obstructed by small trees and underbrush. Rivers with fairly regular alignment and cross section, heavily obstructed by small trees and underbrush.
0.125	Rivers with irregular alignment and cross section, covered with growth of virgin timber and occasional dense patches of bushes and small trees, some logs and dead fallen trees.
0.150-0.200	Rivers with very irregular alignment and cross section, many roots, trees, bushes, large logs and other drift on bottom, trees continually falling into channel due to bank caving.

MANNING ROUGHNESS COEFFICIENTS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by________ Date______
Drawn by______ PLATE 501

MHB-12





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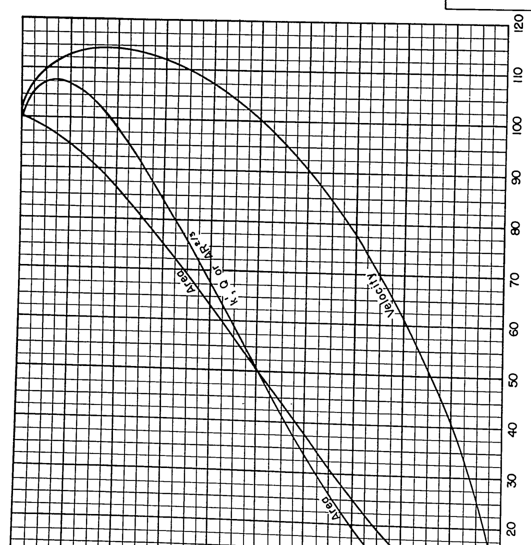
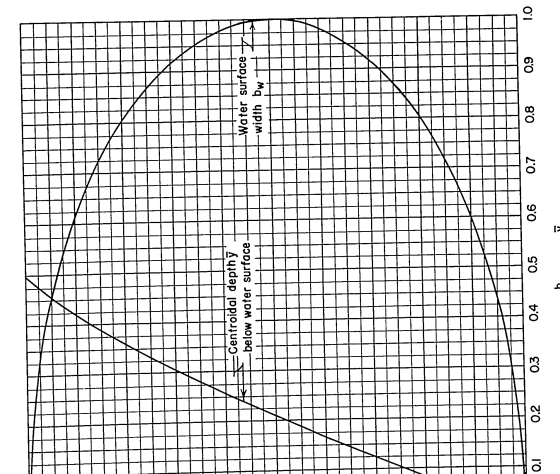
HYDRAULIC CHARACTERISTICS CIRCULAR CONDUITS

MILITARY HYDROLOGY R&D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by______ Date______
Drawn by______



HYDRAULIC CHARACTERISTICS CIRCULAR CONDUITS

MILITARY HYDROLOGY R&D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by______ Date______
Drawn by______



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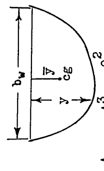
GENERAL SECTION



$h_v = \frac{A}{2b_w}, \quad \frac{A^3}{b_w} = \frac{Q^2}{g}, \quad y_{mean} = \frac{A}{b_w}$

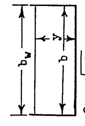
$y_{mean} = \frac{V^2}{g} = \frac{Q^2}{gA^2}$

$V = \sqrt{g y_m} = 5.671 \sqrt{\frac{A}{b_w}}$

$Q = A\sqrt{g y_m} = 5.671\, A \sqrt{\frac{A}{b_w}}$

$H_o = y + \frac{V^2}{2g} = y_c + \frac{y_{mean}}{2}$

RECTANGULAR SECTION



$y_c = \frac{V^2}{g} = \sqrt[3]{\frac{q^2}{g}} = 2/3\ H_o$

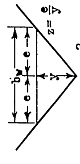
$V = \sqrt{g y_c} = \sqrt[3]{gq}$

$q = \sqrt{g}\ y_c^{3/2}$

$H_o = 3/2\ y_c$

$Q = 5.67\ b\ y_c^{3/2} = 3.087\ b\ H_o^{3/2}$

TRIANGULAR SECTION



Area $A = z y_c^2$

$y_m = 1/2\ y$

$y_c = \frac{2V^2}{g} = 4/5\ H_o = \sqrt[5]{\frac{2Q^2}{gz^2}}$

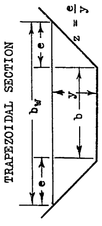
$V = \sqrt{\frac{g y_c}{2}}$

$Q = \sqrt{\frac{g}{2}}\ z y_c^{5/2} = 4.01\ z y_c^{5/2}$

$Q = \sqrt{\frac{g}{2}}\ (4/5)^{5/2}\ z H_o^{5/2}$

$Q = 2.295\ z H_o^{5/2}$

TRAPEZOIDAL SECTION



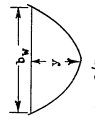
$y_c = \frac{V^2}{g} - \frac{b}{2z} + \sqrt{\frac{V^4}{g^2} + \frac{b^2}{4z^2}}$

$V = \sqrt{\frac{b + z y_c}{b + 2z y_c}\ g y_c}$

$Q = \sqrt{\frac{(b + z y_c)^3 g}{b + 2z y_c}}\ y_c^{3/2}$

$H_o = \frac{(3b + 5z y_c)}{(2b + 4z y_c)}\ y_c$

PARABOLIC SECTION



Area $A = 2/3\, b_w$, $y_m = 2/3\ y$

$y_c = 3/2\ \frac{V^2}{g} = 3/2\ \sqrt[3]{\frac{Q^2}{g b_w^2}}$

$V = \sqrt{2/3\ g y_c}$, $H_o = 4/3\ y_c$

$Q = \sqrt{\frac{8}{27} g}\ b_w y_c^{3/2} = 3.087\ b_w y_c^{3/2}$

$Q = 2.005\ b_w H_o^{3/2}$

HYDRAULIC CHARACTERISTICS PRISMATIC CHANNELS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ____________
Drawn by ____________

DETERMINATION OF THE CONVEYANCE CURVE, THE m' FUNCTION CURVE, AND THE CRITICAL SLOPE CURVE FOR A CANAL OF TRAPEZOIDAL SECTION

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(3) Assumed data.

COMPUTATION OF k', m', and S_c

(4) The depths of Col. 1, Table I were assumed and the area (A), width of water surface (b_w), wetted perimeter (P), and hydraulic radius (R) were computed for each assumed depth and tabulated in Cols. 2, 3, 4, and 5 respectively. To facilitate computations, values of $R^{2/3}$ were computed and listed in Col. 6.

Using the computed values of A, $R^{2/3}$, and the value of n = 0.030 given in the initial data, values of conveyance (k') at each assumed depth were computed by:

$$k' = \frac{1.486}{n} A\ R^{2/3}$$

The computed values of the conveyance were entered in Col. 7.

Using the values of A and b_w computed in Cols. 2 and 3, the ratios A/b_w at each depth were computed and entered in Col. 8. Values of the m' function were computed from Eq. 5-9 as the product of the values in Col. 2 and the square root of the values of Col. 8 and entered in Col. 9.

The squares of the ratios of values of Col. 9 and Col. 7 were listed in Col. 10. The critical slope S_c at each depth was computed from Eq. 5-13 as the product of the values of Col. 10 and the gravitational constant (g) and entered in Col. 11.

HYDRAULIC PROPERTY CURVES

(5) The computed values of Cols. 7, 9, and 11 were plotted against the depth (y) and the conveyance, m', and critical slope curves drawn as shown.

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DETERMINATION OF THE CONVEYANCE CURVE, THE m' FUNCTION CURVE, AND THE CRITICAL SLOPE CURVE FOR A CANAL OF TRAPEZOIDAL SECTION

INITIAL DATA

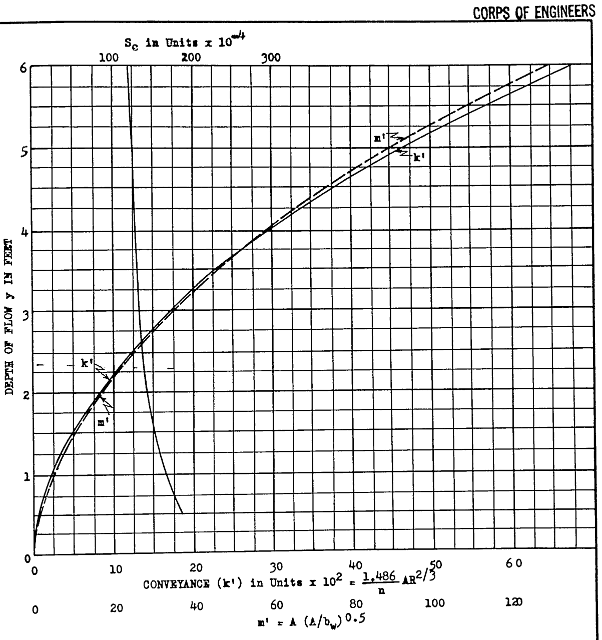
<u>Item</u>

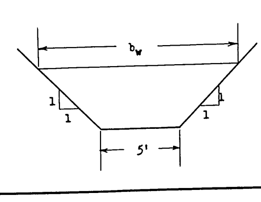
(1) Trapezoidal Channel -- 5' bottom width and 1:1 side slopes.
(2) $n = 0.030$
(3) Bottom Slope $S_o = 0.0001$

(4) TABLE -I

SUMMARY OF COMPUTATIONS FOR k', m', and S_c

y feet	A feet^2	b_w feet	P feet	R = A/P feet	$R^{2/3}$	k' = $\frac{1.486}{n}AR^{2/3}$	A/b_w	m' = $A(A/b_w)^{0.5}$	$(\frac{m'}{k'})^2$	$S_c = g(\frac{m'}{k'})^2$
Col. 1	Col. 2	Col.3	Col. 4	Col. 5	Col.6	Col.7	Col. 8	Col. 9	Col. 10	Col. 11
0.5	2.75	6.0	6.4	0.429	0.569	77.7	.46	1.9	0.000574	0.0185
1.0	6.0	7.0	7.8	0.768	0.838	249.5	.86	5.6	0.000497	0.0160
1.5	9.75	8.0	9.2	1.053	1.037	500.0	1.22	10.8	0.000463	0.0149
2.0	14.0	9.0	10.6	1.312	1.200	834.0	1.56	17.5	0.000440	0.0142
2.5	18.75	10.0	12.1	1.558	1.342	1250.0	1.88	25.7	0.000422	0.0136
3.0	24.0	11.0	13.5	1.783	1.472	1750.0	2.18	35.5	0.000412	0.0133
3.5	29.75	12.0	14.9	2.010	1.585	2340.0	2.48	46.9	0.000400	0.0129
4.0	36.0	13.0	16.3	2.215	1.694	3030.0	2.77	59.9	0.000390	0.0126
4.5	42.75	14.0	17.7	2.410	1.797	3810.0	3.05	74.6	0.000384	0.0124
5.0	50.0	15.0	19.1	2.620	1.898	4700.0	3.33	91.4	0.000378	0.0122
6.0	66.0	17.0	22.0	3.005	2.080	6800.0	3.88	130.0	0.000367	0.0118

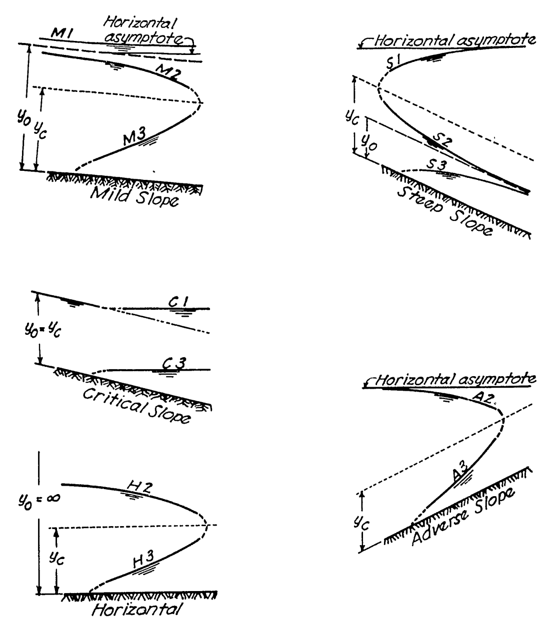




OPEN CHANNEL EXAMPLE
HYDRAULIC PROPERTIES

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by__________ Date__________
Drawn by__________

PLATE 507

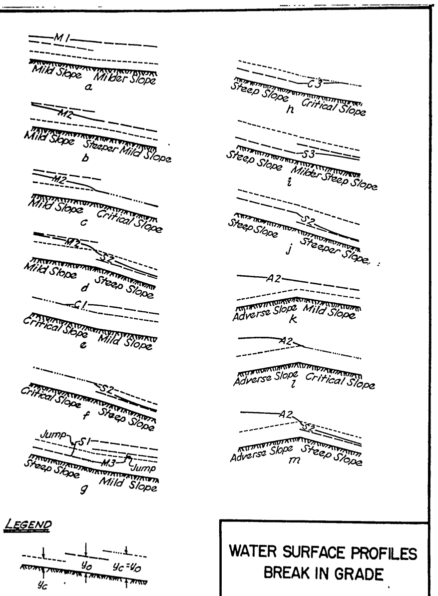


NOTE:
All flow is from left to right.

SURFACE PROFILE CLASSIFICATIONS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ____________
Drawn by ____________

PLATE 508



WATER SURFACE PROFILES
BREAK IN GRADE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by__________ Date__________
Drawn by__________

UNIFORM FLOW IN AN IRREGULAR CHANNEL

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-
(3) Assumed physical data of the irregular channel.

DISCHARGE COMPUTATIONS

(4) The irregular channel was first considered as a single channel and not as a main channel with overbank. The discharge in the open channel was computed as described in the following steps:

1. The areas of the deep and shallow rectangles were computed from the dimensions given in the initial data and entered in Col. 1.
2. The wetted perimeter was computed as the actual wetted length of the sides and bottom (12 + 50 + 10 + 50 + 2) and entered in Col. 2.
3. The hydraulic radius was computed as A/P and entered in Col. 3.
4. The discharge was computed from Eq. 5-5 and entered in Col. 5.

(5) The irregular channel was next divided into two sections as shown by the dotted line on the sketch in item (1). The deep section was considered as the main channel and the shallow section was considered the overbank. The computation of the discharge was the same as for item (4) except the wetted perimeter was computed for each rectangle along the sides, bottom, and dotted line.

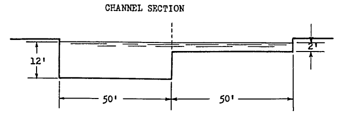
(6) The discharge as computed in item (5) is more correct than item (4).

UNIFORM FLOW IN AN IRREGULAR CHANNEL

INITIAL DATA

Item

(1) CHANNEL SECTION



(2) Bottom slope = 0.0001
(3) Roughness coefficient = 0.020

Item	A feet^2 Col. 1	P feet Col. 2	R feet Col. 3	$R^{2/3}$ Col. 4	Q cfs Col. 5
(4)	700	124	5.65	3.17	1650
(5)	600	74	8.11	4.04	1800
	100	24	1.85	1.51	112
					1912

IRREGULAR CHANNEL EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
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DETERMINATION OF THE BACKWATER CURVE FOR FOURCHE LA FAVE RIVER

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1) The river was divided into reaches in accordance with the methods described in Par. 82. The mean cross section of each reach was determined and the conveyance curve plotted as a function of the water surface elevation by the procedures described in Par. 70.

(2) The discharge was assumed as given in the initial data.

(3) The starting elevation at mile 1.5 was assumed to be 251.0 feet msl.

BACKWATER COMPUTATION

(4) The water surface profile up the Fourche La Fave River was computed as described in the following steps:

1. The channel mileage at the ends of the reaches (as measured along the thalweg, or center of mass if of flowing water) was entered in Col. 1.
2. The length of each reach was determined from the channel mileage of Col. 1 and entered in Col. 2.
3. The mean cross section of the reach was determined and its mileage, as measured along the thalweg, was entered in Col. 3.
4. The proportionate fall in the water surface from the mean cross section to the lower end of the reach was determined as the ratio of the length of reach below the mean cross section, to the total length of reach, and entered in Col. 4.
5. The starting water surface elevation at the lower end of the lower reach was determined from a known rating curve and entered in Col. 10, Line 1.
6. A water surface elevation was assumed at the mean cross section in the first reach and entered in Col. 5, Line 2.
7. The conveyance was determined from the conveyance curve for the elevation assumed in Col. 5 and entered in Col. 7, Line 2.
8. The fall in the entire reach was determined as the product of the reach length (given in Col. 2) and the term $(Q/k')^2$. The value of the conveyance was given in Col. 7, Line 2. The fall was entered in Col. 8, Line 2.
9. The proportionate fall of the water surface from the mea mean cross section to the lower end of the reach was determined as the product of the factor of Col. 4, Line 2, and the fall in the reach of Col. 8, Line 2, and entered in Col. 9, Line 2.
10. The computed water surface at the mean cross section was determined as the sum of the proportionate water surface fall of Col. 9, Line 2, and the water surface elevation at the lower end of the reach given in Col. 10, Line 1, and was entered in Col. 6, Line 2.
11. The assumed water surface elevation in Col. 5 was compared with the computed water surface elevation of Col. 6. If the elevations are different by an amount greater than the degree of accuracy desired, another trial is made and the above steps repeated. Normally a water surface profile is computed to the nearest tenth of a foot.
12. After the assumed and computed water surfaces were brought into a reasonable balance, the water surface elevation at the upper end of the reach was determined as the sum of the total fall in Col. 8, Line 2, and the water surface elevation at the lower end of the reach in Col. 10, Line 3.
13. A new water surface elevation was assumed in Col. 5, Line 4, for the next reach and steps 6 and 12 repeated. The water surface elevation of the upper end section of the first reach as given in Col. 10, Line 3, became the water surface elevation of the lower end section of the second reach.
14. Steps 3 to 13 were repeated for each reach of the river and the waters surface profile determined from Col. 1 and Col. 10.

DETERMINATION OF THE BACKWATER CURVE FOR FOURCHE LA FAVE RIVER

COMPUTATION SHEET

Item

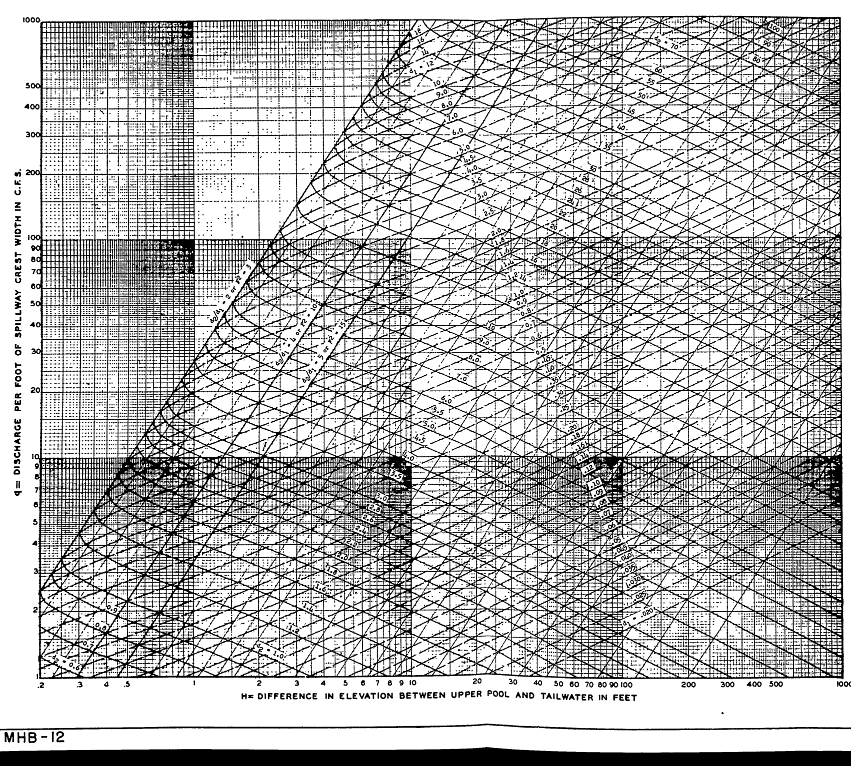
INITIAL DATA

(1) Fourche La Fave river cross sections from mile 3.1 to 3.6
(2) Discharge constant in the length of river considered = 5,000 cfs
(3) Starting elevation at the lower end of the first reach of the river equal to 251.0 ft. msl.

TABLE I
BACKWATER COMPUTATIONS

Line	Channel Mileage of End Sections Along Thalweg	Reach Length (ft)	Channel Mileage of Mean Section	Factor	Water Surface Elevation at Mean Section (feet msl.) Trial	Computed	Conveyance of Mean Section (1000s)	W.S. Fall in Reach $(Q/k')^2L$ (ft.)	Factor times Fall in Reach	W. S. Elevation at Upper end of Section (ft. msl.)
	Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10
1	1.50									251.0
2		9130	3.10	0.928	254.80	254.85	235	4114	3.85	
3	3.23									255.14
4		1600	3.36	0.437	255.40	255.41	252.4	0.63	0.27	
5	3.53									255.77
6		4300	3.68	0.186	255.85	255.85	492	0.44	0.08	
7	4.34									256.21

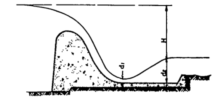
BACKWATER EXAMPLE
NATURAL CHANNEL



FORMULA

$$H = \frac{q^2}{2gd_1^2} \quad + \quad \cdot \quad d_1 \qquad\qquad - d_2$$

$$H = \frac{2q^2}{gd_2^2\left[\sqrt{\frac{8q^2}{gd_2^3}+1}-1\right]^2} + \frac{d_2}{2}\left[\sqrt{\frac{8q^2}{gd_2^3}+1}-1\right] - d_2$$



H = Difference in elevation between upper pool and tailwater in feet.

q = Discharge per foot of spillway crest width in c.f.s.

d_1 = Depth above the hydraulic jump in feet.

d_2 = Depth below the hydraulic jump in feet.

g = Acceleration due to gravity, 32.2 feet/second^2

$F^2 = \frac{q^2}{gd_1^3}$

NOTES

The diagram does not include friction losses. Compensation for friction may be made by subtracting the equivalent friction head from H, the difference between upper and lower pool elevations, before entering into the diagram.

The lines $\frac{d_2}{d_1}$ = 2, 4 and 5, correspond to F^2 values of 3, 10, and 15, respectively.

$F^2 = 3$ or $\frac{d_2}{d_1} = 2$, marks the boundary between jumps of the undular and shock types.

Model studies indicate that a strong hydraulic jump forms in the region to the right of $F^2 = 12$, or $\frac{d_2}{d_1} = 4.42$

REQUIREMENTS FOR HYDRAULIC JUMP BELOW SPILLWAYS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date __________
Drawn by __________

DETERMINATION OF THE SPECIFIC ENERGY CURVE AND THE M(y) CURVE FOR A TRAPEZOIDAL CANAL

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(3) The initial data was taken from the sketch, and the discharge was assumed to be 300 cfs. The critical depth was determined from Plate 507 to be 3.74 ft.

COMPUTATION OF H_o AND M(y) CURVES

(4) The specific energy and M(y) function curves were computed as described in the following steps:

1. The canal depths were assumed as shown in Col. 1, Table I.
2. The cross-sectional area of flow was computed for each depth in Col. 1 and entered in Col. 2.
3. The velocities and velocity heads were computed for a constant discharge of 300 cfs and entered in Cols. 3 and 4, respectively.
4. The specific energy was determined as the sum of the depths and velocity heads given in Cols. 1 and 4 and entered in Col. 5.
5. The M(y) function was determined from equation 5-33:

$$M(y) = \frac{Q^2}{Ag} + A\bar{y}$$

6. The first term of the right hand side of equation 5-33 was computed for a discharge of 300 cfs:

$$\frac{300^2}{32.2A} = \frac{2795}{A}$$

The ratio 2795/A was computed for each area of Col. 2 and listed in Col. 6.

7. The centroidal depth of each flow section for the depths given in Col. 1 was computed from the equation taken from Fig. 2, Plate 101:

$$\bar{y} = \frac{y\,(b_w + 2b)}{3\,(b_w + b)}$$

$$\bar{y} = \frac{y\,(b_w + 10)}{3\,(b_w + 5)}$$

and entered in Col. 7.

8. The second term of the right hand side of equation 5-33 was computed as the product of the values listed in Cols. 2 and 7 and entered in Col. 8.
9. The value of the M(y) function for each depth was determined as the sum of the values in Cols. 6 and 8 and entered in Col. 9.

H_o AND M(y) CURVES

(5) The computed values of Cols. 5 and 9 were plotted against the depths given in Col. 1, and the H_o and M(y) curves drawn as shown.

DETERMINATION OF THE SPECIFIC ENERGY CURVE AND THE M(y) CURVE FOR A TRAPEZOIDAL CANAL

Item

INITIAL DATA

(1) Trapezoidal canal 5.0 ft. bottom width and 1:1 side slopes.

(2) Discharge $Q = 300$ cfs

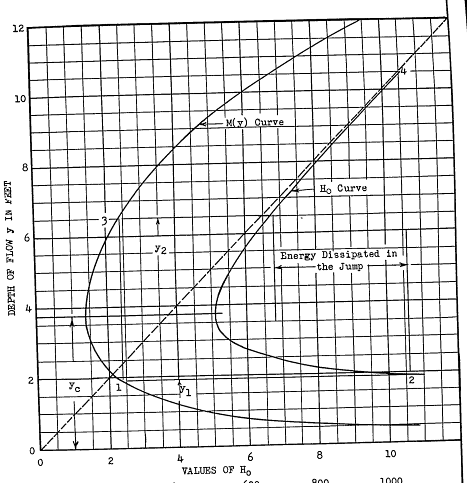
(3) Critical depth = 3.74 ft.

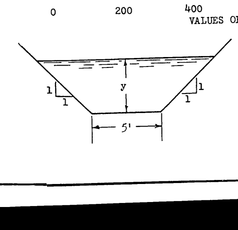
(4) TABLE I

COMPUTATION OF H_o AND M(y) CURVES

y feet Col. 1	A sq.feet Col. 2	V feet Col. 3	$V^2/2g$ feet Col. 4	H_o Col. 5	2795/A Col. 6	$\bar{y}$ feet Col. 7	$A\bar{y}$ Col. 8	M(y) Col. 9
0.50	2.75	109.0	185.0	185.5	1,017.0	0.242	0.66	1,018.0
1.00	6.00	50.0	38.9	39.9	466.0	0.471	2.83	469.0
2.00	14.00	21.4	7.14	9.14	200.0	0.905	12.70	213.0
3.00	24.00	[illegible]	2.43	5.43	116.0	1.315	31.50	147.5
3.50	29.75	10.11	1.58	5.08	94.0	1.51	44.90	139.0
3.74	32.69	9.08	1.31	5.05	85.5	1.60	52.30	137.8
4.00	36.00	8.33	1.08	5.08	77.7	1.71	61.50	139.2
5.00	50.00	6.00	0.56	5.56	55.9	2.08	104.0	160.0
6.00	66.00	4.55	0.32	6.32	42.3	2.46	162.3	205.0
8.00	104.00	2.88	0.13	8.13	26.9	3.18	331.0	358.0
10.00	150.00	2.00	0.06	10.06	18.6	3.89	584.0	603.0
12.00	204.00	1.47			13.7	4.58	945.0	959.0

Item (5)





HYDRAULIC JUMP EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
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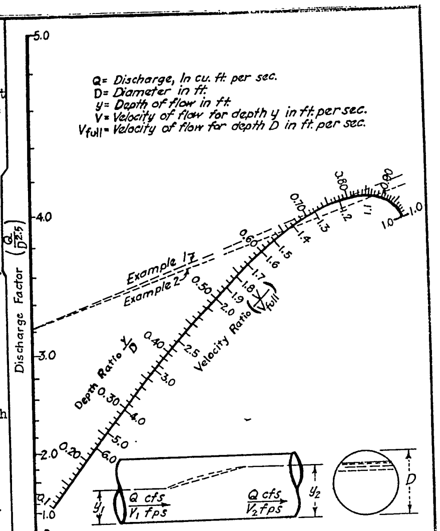
To use the diagram for a given quantity Q, flowing through a pipe of diameter D, the discharge factor, $Q/D^{2.5}$, is calculated first. A straight line is then drawn through the corresponding point on the discharge factor scale to intersect the scale of depth-ratios, y/D. One of three conditions then exists.

1. The straight line is tangent to the depth-ratio scale, the point of tangency being the value y/D corresponding to the critical depth. At the point of tangency one can also read the velocity ratio for that flow.

2. If the straight line intersects the curve at two points as a secant, the water will jump from the depth and velocity corresponding to the lower depth-ratio to the depth and velocity corresponding to the higher of the two depth-ratios.

3. The third case is that where the straight line cuts the curve only once, indicating that the pipe will flow full below the hydraulic jump. When this occurs air must be supplied to the conduit or the phenomenon will become merely a sudden expansion and cease to behave as a hydraulic jump. The following numerical examples illustrate the use of the diagram:



Example 1. Find the critical depth and critical velocity in a circular pipe 3 ft. in diameter discharging 50 cfs. $Q/D^2 = 50/(3)^2 = 5.55$; $V_{full} = 5.55/0.785 = 7.07$ ft. per sec. ; $Q/D^{2.5} = 5.55/\sqrt{3} = 3.21$. A straight line through discharge factor 3.21 on the chart is tangent to the curve at depth ratio 0.77. Hence, the critical depth, y_c, is 0.77 x 3.0 = 2.3 ft. The corresponding velocity ratio is 1.21 and the critical velocity, V_c, is 1.21 x 7.07 = 8.55 ft. per sec.

Example 2. A discharge of 50 cfs. flows at a depth of 2.0 ft. in a circular pipe 3 ft. in diameter. Find the velocity of flow above the hydraulic jump and the depth and velocity of flow below the jump. From Example 1, the velocity is 7.07 ft. per sec and $Q/D^{2.5} = 3.21$. Above jump, $y_1/D = 2/3 = 0.67$. From the chart, when $y_1/D = 0.67$, $V_1/V_{full} = 1.4$ and velocity at Sec. 1 is 1.4 x 7.07 = 9.9 ft. per sec. A straight line through $Q/D^{2.5} = 3.21$ and $y_1/D = 0.67$ intersects the curve at $y_2/D = 0.88$ and $V_2/V_{full} = 1.08$. Depth of flow below jump, $y_2 = 0.88 \times 3.0 = 2.64$ ft. Velocity below jump, $V_2 = 1.08 \times 7.07 = 7.65$ ft. per sec.

HYDRAULIC JUMP NOMOGRAPH

MILITARY HYDROLOGY R & D BRANCH
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Prepared by ________ Date ________
Drawn by ________
PLATE 514

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DETERMINATION OF THE WATER SURFACE
PROFILE OF A PRISMATIC CHANNEL

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(7) Assumed physical data of the large outlet conduit of a dam.

(8) The area, conveyance, and m' function curves were computed for the circular conduit in the same manner as described in paragraphs 70 and 72, respectively, and also as shown by the example on Plate 507.

(9) The M(y) function curve was computed for a discharge of 833 cfs as described in paragraph 91 and as shown by the example on Plate 513.

(10) The normal depth was determined by use of equation 5-23 and the conveyance curve as follows:

$$k' = Q/S_o^{0.5}$$

$$= \frac{833}{(0.01145)^{0.5}}$$

$$k' = 7790$$

Enter the conveyance curve with k' = 7790 and read the depth, therefore:

$$y_o = 3.6 \text{ ft.}$$

(11) The critical depth was determined by use of equation 5-10 and the m' function curve as follows:

$$m' = Q/(g)^{0.5}$$

$$= \frac{833}{5.67}$$

$$m' = 147$$

Enter the conveyance curve with m' = 147 and read the depth, therefore:

$$y_c = 5.6 \text{ ft.}$$

(12) A qualitative analysis of the water surface profile was made, based on the criteria given in paragraph 77 as described in the following steps:

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1. From items (10) and (11) above, it was determined that the normal depth was below the critical depth for the discharge considered.

2. Since the normal depth was less than the critical depth, the bottom slope must be greater than the critical slope and the profiles must be of the steep type or S series.

3. The initial depth at the outlet portal was computed as the difference between items (4) and (5) from the initial data on the computation sheet.

4. Since the initial profile depth was considerably above both the normal and critical depths, the water surface profile would be an S1 curve as shown in Fig. (g), Plate 509.

5. Since the bottom slope of the conduit was greater than the critical slope, the water surface at the conduit inlet would pass through the critical depth at the break in grade (station 8 + 73).

6. The profile depth at the break in grade was between the normal depth and the critical depth; therefore, the water surface profile was an S2 curve.

7. The S2 profile with a depth less than critical depth, overlaps the S1 profile with a depth greater than critical depth, forming the hydraulic jump where the sequent depth of the S2 curve intersects the S1 curve.

S1 WATER SURFACE PROFILE COMPUTATION

(13) The S1 water surface profile was computed for the 22 ft. diameter conduit with a constant discharge of 833 cfs as described in the following steps:

1. The initial depth of water at the conduit outlet was entered in line 1, Col. 1.

2. The cross-sectional area for the initial depth of flow was determined from the area curve and listed in Col. 2.

3. The velocity and velocity head was computed from the discharge (833 cfs) and the cross-sectional area of step 2 and entered in Col. 3 and 4, respectively.

4. The specific head was computed as the sum of the depth of Col. 1 and the velocity head of Col. 4 and entered in Col. 5.

5. The conveyance (k') was determined from the conveyance curve at the depth given in Col. 1 and entered in Col. 6.

6. A small increment in the reduction of depth was assumed and entered in line 2, Col. 1.

7. The upstream depth of the reach was determined as the difference between the initial depth in line 1, Col. 1 and the increment of depth in line 2, Col. 1, and entered in Line 3, Col. 1.

8. The hydraulic properties were computed for the upstream depth in the same manner as steps 2 through 5 and entered along line 3, in their respective columns.

9. The change in specific head was determined as the difference in lines 1 and 3, Col. 5, and entered in line 2, Col. 5.

10. The mean value of the conveyance in the reach was determined as the average of the conveyances of lines 1 and 3, Col. 6, and entered in line 2, Col. 6.

11. The energy gradient was determined from equation 5-6 as the square of the ratio of the discharge and the mean conveyance and entered in line 2, Col. 7.

12. The difference between the bottom slope and energy gradient was entered in line 2, Col. 8.

13. The reach length for the assumed increment of change in depth was computed by equation 5-28 and entered in line 2, Col. 9. The reach length was determined as the ratio of the change in specific head of line 2, Col. 5, and the difference in slopes of line 2, Col. 8.

14. The reach length of Col. 9 was added to the station number of the lower end of the reach to give the station of the upper section of the reach.

15. The invert elevation of Col. 11 was computed as the sum of the invert elevation at the conduit outlet and the product of the bottom slope and reach length, and entered in line 3, Col. 11.

16. The water surface elevation at the end of the reach was computed as the sum of the invert elevation of Col. 11 and the depth of flow in Col. 1, and entered in line 3, Col. 12.

17. A new increment in the change in depth is assumed.

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entered in line 4, Col. 1.

18. The new upstream depth was computed for the second reach as described in step 7 and entered in line 5, Col. 1.

19. The hydraulic properties of the section were computed in the same manner as steps 2 through 5 and entered on line 5, in their respective columns.

20. The reach length and water surface elevation was computed in the same manner as in steps 9 through 16.

21. The profile was continued upstream until the depth was reduced to the critical depth which is the limit of the S1 curve.

S2 WATER SURFACE PROFILE AND SEQUENT DEPTH COMPUTATION

(14) The S2 water surface profile was computed in the same manner as the S1 profile except the initial depth was the critical depth and the computation proceeded in the downstream direction. The depth of flow decreases in the downstream direction of flow for the S2 profile curve. It should be noted that the change in specific head in equation 5-28 is the downstream specific head minus the upstream specific head. The location of the hydraulic jump was determined as follows: (1) Enter the M(y) curve with the initial depth given in line 7, Col. 1, and read the sequent depth in the upper segment of the curve. This sequent depth was entered in line 7, Col. 13. (2) The elevation of the sequent depth was determined as the sum of the invert elevation of line 7, Col. 11 and the sequent depth, and entered in line 7, Col. 14. (3) The sequent depths for each flow depth of Col. 1 were determined in the same manner and entered in Col. 14.

WATER SURFACE AND SEQUENT DEPTH PROFILES

(15) The bottom slope profile was plotted to an exaggerated scale from the values of the stations and bottom elevation of Col. 10 and Col. 11. The S1 water surface profile was plotted from the values in Col. 10 and Col. 12 of Table I. The S2 water surface profile was plotted from the values in Col. 10 and Col. 12 of Table II. The sequent depth profile was plotted from the values in Col. 10 and Col. 14 of Table II. The point at which the S1 profile and the sequent depth profile intersect determined the point of the hydraulic jump which was at station 7+95. The length of the jump was shown as about four times the depth below the jump.

DETERMINATION OF THE WATER SURFACE PROFILE OF A PRISMATIC CHANNEL

Item

INITIAL DATA

(1) A circular concrete conduit 22 feet inside diameter.

(2) Length of conduit = 873 ft.

(3) Slope of conduit = 0.01145

(4) Invert elevation at conduit outlet = 1219.0 ft. msl.

(5) Tailwater elevation = 1235.7

(6) Discharge = 833 cfs

(7) Roughness coefficient = 0.013

(8) Area, Conveyance, and m' funct on curves.

(9) M(y) function curve.

(10) Normal depth = 3.6 ft.

(11) Critical depth = 5.6 ft.

(12) The profile will follow an M2 curve from the reservoir water surface to the break in grade at Station 8 + 73. Downstream from the break in grade the profile will follow an S2 curve to a point where the hydraulic jump will form. The water surface will then flow out of the conduit with an S2 curve to meet the tailwater at elevation 1235.7 ft. msl.

Item

(13)

Line	Dep ft. Col.
1	16.
2	0.
3	16.
4	1.
5	15.
6	1.
7	14.
8	1.
9	13.
10	1.
11	12.
12	1.
13	11.
14	1.
15	10.
16	1.
17	9.
18	1.
19	8.
20	1.
21	7.
22	1.
23	6.
24	1.
25	5.

(14)

Line	Depth ft. Col.1	Ar f Co
1	5.6	7
2	0.6	
3	5.0	6
4	0.5	
5	4.5	5
6	0.5	
7	4.0	4

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Item

(13)

TABLE I

S1 WATER SURFACE PROFILE COMPUTATIONS

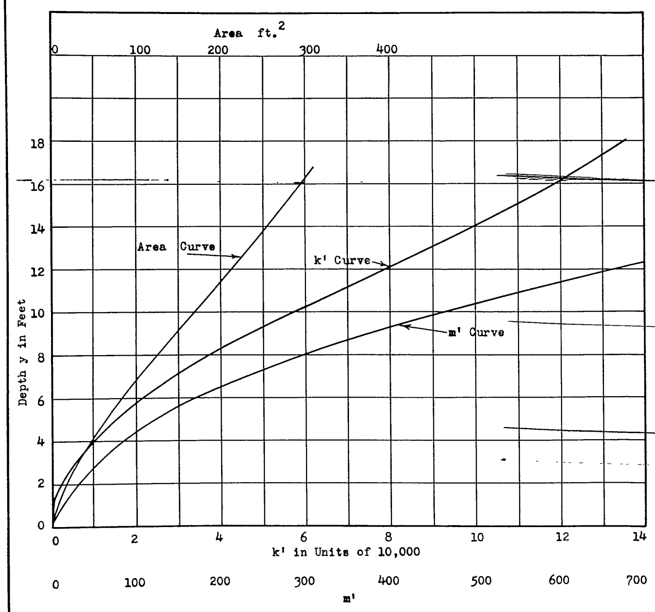
Line	Depth ft. Col. 1	Area ft.^{2} Col. 2	Velocity ft/sec Col. 3	$V^2/2g$ ft. Col. 4	H_o ft. Col. 5	$k' \times 10^{-4}$ Col. 6	$S \times 10^4$ Col. 7	$S_o - S \times 10^4$ Col. 8	$\triangle L$ ft. Col. 9	Station Col. 10	Invert Elevation ft.msl. Col. 11	W.S. Elev. ft. msl. Col. 12
1	16.7	309	2.70	0.113	16.813	12.45				0 + 00	1219.0	1235.7
2	0.7				0.689	12.15	0.47	114.08	60			
3	16.0	295	2.82	0.124	16.124	11.85				0 + 60	1219.69	1235.69
4	1.0				0.982	11.38	0.54	114.01	86			
5	15.0	276	3.02	0.142	15.142	10.90				1 + 46	1220.67	1235.67
6	1.0				0.977	10.41	0.585	113.97	86			
7	14.0	255	3.26	0.165	14.165	9.92				2 + 32	1221.66	1235.66
8	1.0				0.966	9.43	0.78	113.77	85			
9	13.0	233	3.58	0.199	13.199	8.94				3 + 17	1222.63	1235.63
10	1.0				0.959	8.42	0.98	113.57	84			
11	12.0	212	3.93	0.240	12.240	7.90				4 + 01	1223.59	1235.59
12	1.0				0.950	7.35	1.28	113.27	84			
13	11.0	191	4.35	0.290	11.290	6.80				4 + 85	1224.56	1235.56
14	1.0				0.907	6.28	1.76	112.79	81			
15	10.0	168	4.96	0.383	10.383	5.75				5 + 66	1225.48	1235.48
16	1.0				0.885	5.22	2.55	112.00	79			
17	9.0	147	5.66	0.498	9.498	4.68				6 + 45	1226.39	1235.39
18	1.0				0.796	4.19	3.96	110.59	72			
19	8.0	124	6.72	0.702	8.702	3.70				7 + 17	1227.21	1235.21
20	1.0				0.704	3.30	6.38	108.17	65			
21	7.0	104	8.01	0.998	7.998	2.90				7 + 82	1227.96	1234.96
22	1.0				0.431	2.55	10.7	103.85	42			
23	6.0	83	10.04	1.567	7.567	2.20				8 + 24	1228.44	1234.44
24	1.0				0.101	2.05	16.5	98.05	10			
25	5.6	76	10.96	1.866	7.466	1.90				8 + 34	1228.55	1234.15

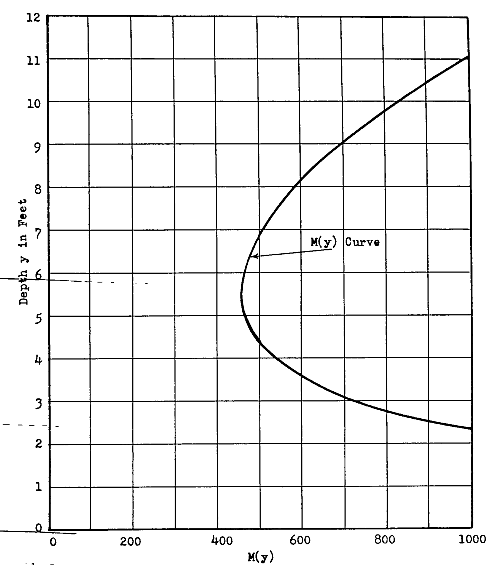
(14)

TABLE II

S2 WATER SURFACE PROFILE AND SEQUENT DEPTH COMPUTATIONS.

Line	Depth ft. Col.1	Area ft.^{2} Col.2	Velocity ft/sec Col. 3	$V^2/2g$ ft. Col. 4	H_o ft. Col. 5	$k' \times 10^{-4}$ Col. 6	$S \times 10^4$ Col. 7	$S_o - S \times 10^4$ Col. 8	$\triangle L$ ft. Col. 9	Station Col. 10	Invert Elevation ft. msl. Col. 11	W.S.Elev. ft. msl. Col. 12	Sequent Depth ft. Col.13	WS El. Seq.Dp ' msl Col.14
1	5.6	76	10.96	1.87	7.47	1.90				8 + 73	1229.00	1234.60		
2	0.6				0.08	1.71	23.7	90.8	9					
3	5.0	65	12.81	2.55	7.55	1.52				8 + 64	1228.90	1233.90	6.10	1235.0
4	0.5				0.51	1.38	36.4	78.1	65					
5	4.5	55	15.15	3.56	8.06	1.25				7 + 99	1228.16	1232.66	6.70	1234.86
6	0.5				1.04	1.10	57.2	57.3	192					
7	4.0	46	18.11	5.10	9.10	0.95				6 + 07	1225.96	1229.96	7.45	1233.41





PRISMATIC CHANNEL EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ________
Drawn by ____________

CORPS OF ENGINEERS

DEPARTMENT OF THE ARMY

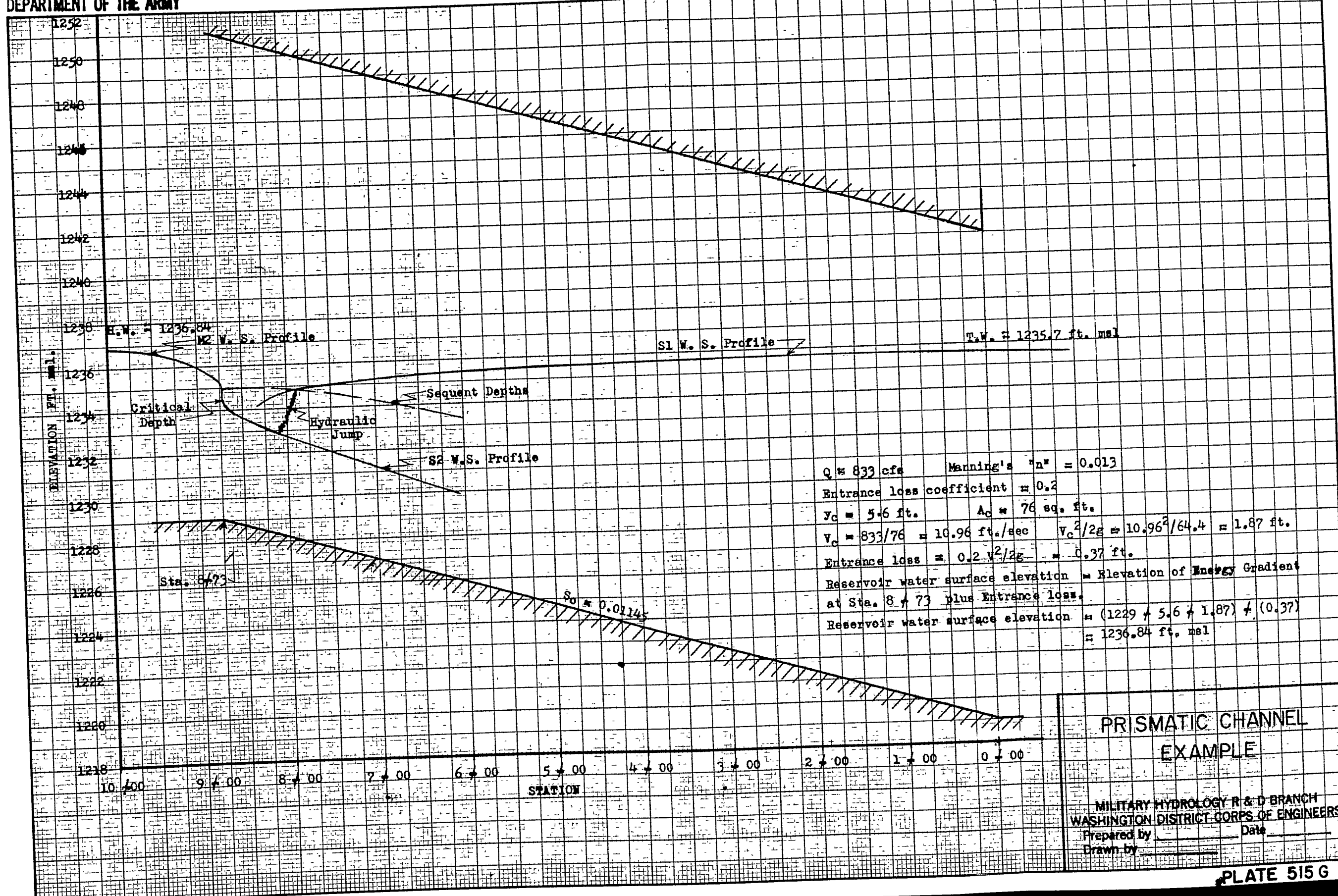


PLATE 515 G

MHB-12

CHAPTER VI
SPILLWAYS

SECTION A: BASIC CONSIDERATIONS

94. Function of Spillways. The primary function of a spillway is to release surplus water from a reservoir in order to prevent overtopping of the dam. Spillway structures are designed to provide sufficient discharge capacity to pass the spillway design flood, as modified by available reservoir storage, without exceeding the maximum permissible water surface elevation in the reservoir.

95. Types of Spillways. The most common types of spillways may be classed as ogee, chute, side-channel, morning-glory, and siphon. Other less common types include free overhand, stepped, and partial morning-glory. Any of these types, except siphons, may have gated or ungated crests.

96. Component Parts of Spillways. The component parts of spillways include approach channel, weir or control section, discharge channel, apron or stilling basin, and outlet channel.

97. Discharge Capacity. a. The discharge capacity of a spillway is normally computed by a basic weir equation or an orifice equation.

b. Weirs. The discharge over a spillway that has a weir for a control section is usually computed by an equation of the form:

$$Q = C_q L H^{1.5} \qquad (6\text{-}1)$$

where

Q = discharge in cfs
H = total head on the crest (including the approach velocity head) in feet
L = the effective length of spillway in feet
C_q = a variable discharge coefficient

C_q is a function of the channel approach depth, slope of the upstream and downstream face of the spillway, the ratio of the actual head to the design head on the spillway, the degree of submergence, and the shape of the spillway crest.

c. Orifice. The discharge of a siphon spillway usually is computed by an orifice type equation of the form:

$$Q = C_q A \ (2gH)^{0.5} \qquad (6\text{-}2)$$

where

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Q = discharge in cfs,
H = head on the siphon equal to the difference in elevation between head water and the tail-water or center line of outlet, whichever gives the lesser head. (A siphon has a limiting effective head of approximately 30 feet at sea level).
A = the cross-sectional area of the throat of the siphon in square feet.
C_q = the coefficient of discharge which varies between wide limits, 0.3 to 0.9, depending on the design of the particular siphon.

98. Design Head. a. The design head on a spillway crest is defined as the maximum head under which the spillway is expected to operate. If the spillway profile is designed to fit the lower nappe of a sharp-crested weir,the design head is the head at which the pressure on the downstream face is zero. Heads greater than the design head would cause negative pressures on the downstream face and would have discharge coefficients higher than the design discharge coefficient. Heads lower than the design head would cause positive pressures and have lower discharge coefficients. A great many spillways are not designed to fit the lower nappe of a sharp-crested weir, but for economic, construction, structural, or hydraulic design reasons are designed as compound circles or with exponential curvatures. These spillway profiles often approximate the shape of the lower nappe of a sharp-crested weir by varying degrees.

b. Knowing the design head and profile of a spillway, the design coefficient would be determined by model studies or by comparison with known coefficients for dams with similar shapes and heads. During military field operations, when the principals of military hydrology are applied, it is probable that the design head on a spillway would not be known. Therefore, it is necessary to devise a means of approximating the design head and the coefficient of discharge.

c. Spillway Crest Profile. One method of approximating the design head on a spillway would be to make an anslysis of the spillway crest profile. The radius of a circular crest spillway or the general shape of an ogee spillway has a definite relationship to the design head for conditions of zero pressure on the downstream face. Also the ratio of the crest rise to the design head is a constant for an ogee spillway profile designed for zero pressure at the design head.

(1) Circular Crests. A preliminary study has been made to determine the relationship between the radius of a circular spillway crest and the design head. A number of circular crest profiles were plotted for different length radii as shown on Plate 601. The radii were matched with the crests of dams for known design heads, radii, and coefficients of discharge. It was found that a reasonable degree of accuracy would be obtained by assuming the radius of curvature to be the design head. The design head as determined in this manner

would be used to determine the discharge coefficient as will be explained in a later paragraph.

(2) Ogee Crests. A method of determining the design head on spillway crests that approximate the shape of the lower nappe of a sharp-crested weir is to match the given spillway crest profile with the lower nappe profiles of known design head. Such profiles were plotted on Plate 601 for the design heads indicated on each curve. A spillway crest profile would be drawn on transparent paper to the same scale, and matched with the profiles on Plate 601. The head shown on the matched profile is estimated design head.

(3) Crest Rise. Another method of estimating the design head on the spillway would be to determine the crest rise, and divide that rise by a constant factor. Plot the cross section of the spillway section of the dam. Determine the difference in elevation in feet between the spillway crest and the point of intersection of the vertical upstream face and the crest profile. Divide the crest rise by 0.126. This quotient is approximately the design head on an ogee crest with a vertical upstream face, see Plate 602.

d. Physical Dimensions of Spillway. (1) One method of approximating the design head on a spillway crest would be to make an analysis of the physical dimensions of the spillway cross section. Frequently, the spillway on a large dam will be gated and have a bridge over the spillway crest. The low steel or concrete of the bridge, the lop of the gates when in a fully raised position, the trunnion pins, and the spray walls will normally be set a small distance above the upper nappe profile corresponding to the spillway design flood. From these factors an estimate can be made of the design head.

(2) Assume a design head on the spillway crest and compute the upper and lower nappe profiles of a sharp-crested weir by the methods given in Par. 26. From the computed profiles, measure the nappe thickness along and normal to the lower nappe profile. Plot the nappe thickness normal to the downstream face of the spillway profile; and draw the estimated design water surface profile. This profile should be below the bridge low steel, gate lip, and gate trunnions by a few feet. Several trials may be needed to approximate the design head by this criteria.

e. Freeboard. The design head on the dam is also estimated from the freeboard. Freeboard is defined as the difference in elevation between the top of the dam, and the maximum reservoir level that would be attained during the spillway design flood. The net freeboard is the sum of the heights of wind tide, seiches, and wave height including ride-up on the upstream face of the dam, plus a small margin of safety in feet based on judgment. Earth dams, being more vulnerbale to destruction from overtopping, require more freeboard for safe design than concrete dams. Rockfill dams, although more resistant to wave-splash over the crest than earth dams, will normally have about the same safety factor for freeboard as an earth dam. The magnitude of the wind tide, seiches, and wave heights occurring on a dam is computed as follows:

(1) Wind Tide. The ZUIDER ZEE Formula /1/ is used to compute the wind tide, thus:

$$h_t = \frac{V^2L}{1400y} \text{ Cos } \theta \qquad (6\text{-}3)$$

where

h_t = wind tide in feet above the still pool
V = wind velocity in miles per hour
L = fetch or straight length of open water above the dam in statute miles subject to wind action
y = average approach depth of water in feet
θ = angle between wind direction and fetch length

(2) Wave Height Including Ride Up. The Molitor formula /2/ for wave heights is as follows:

$$h_w = 0.17\ (VL)^{0.5} + 2.5 - L^{0.25} \qquad (6\text{-}4)$$

where

h_w = height of waves from trough to crest in feet
V = wind velocity in miles per hour
L = fetch length in statute miles

The height of wave ride up above the still pool is approximately equal to 1.5 h_w /3/. This height would be added to the height of wind tide. Equation 6-4 is solved diagrammatically on Plate 602.

(3) Seiches. A periodic undulation is called a seiche, and is caused by intermittent wind action, variations in atmospheric pressure or earthquakes. Amplitudes of 0.5 foot or more readily occur in reservoirs of moderate size. Seiches have occurred in Lake Geneva, Switzerland of over 6.0 feet. The factor of safety used in computing freeboard should be sufficient to cover possible seiches. Normal engineering design practice specifies a minimum 3-foot freeboard for masonry dams and a 5-foot freeboard for earth dams. The freeboard computed from the above formulas, including a small safety factor in feet, would be subtracted from the elevation of the crest of the dam to give the estimated design water surface elevation. The elevation of the spillway crest subtracted from the design water surface elevation will give the approximate design head.

f. Estimate of the Design Head. After estimating the design heads from the above methods, a final design head is selected for the spillway.

99. Effective Crest Length. The effective length of the spillway to be used in the discharge equation is computed as follows:

$$L = L' - KNH \qquad (6\text{-}5)$$

where

L = the effective crest length of spillway
L' = the clear opening of the spillway between the gate piers
K = the pier contraction coefficient
N = the number of such contractions (2 for each gate bay)
H = the total head on the crest including the approach velocity head

The contraction coefficient /4/ is a function of the ratio of the actual head to the design head, and the pier nose shape as shown on Plate 603. Also the coefficient is a function of the pier length, see Plate 604, and the approach depth as shown on Plate 605.

SECTION B: OGEE SPILLWAYS

100. Definition. An ogee spillway crest has an S-shaped or reverse curved profile for its downstream face. The crests of efficient high overfall ogee spillways are shaped to con form closely to the profile of the lower nappe of a sharp-crested weir when the flow corresponds to the design head, thus producing low-pressures on the crest. Plate 606 depicts an example of an ogee spillway.

101. Discharge Capacity. The discharge capacity, design head, and effective crest length of an ogee spillway are discussed in paragraphs 97 and 98.

102. Discharge Coefficient. a. The coefficient of discharge C_q is a function of the depth of approach channel, the ratio of actual head to design head, the slopes of the upstream and downstream faces of the spillway, and the degree of submergence. The individual effects of these factors have been investigated; and the resulting data is generally accurate within reasonable limits. It should be noted, however, that the combined result, if determined from individual effects, may be considerably in error. It is desirable to obtain the discharge coefficients from models that are properly designed by the model laws for each prototype structure.

b. Depth of Approach Channel and Slope of Upstream Face. The curves shown on Plate 607 give values for the design discharge coefficient /5/ as a function of the approach depth, and the angle between the upstream face and the vertical. It should be noted that the curves are based on a relatively low approach depth with respect to the design head and that the downstream slope is steep as it is based on the free overfall of a sharp-crested weir. The value of C_q shown on Plate 607 ranges from about 4.0 to 3.5.

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c. Downstream Face Slope. The effect of the angle of the downstream spillway face on the coefficient of discharge is shown on Plate 608. The coefficient of discharge /6/ was plotted as a function of the head on the crest of the spillway; and the angle between the downstream face of the spillway, and a horizontal plane. The curves were taken from data of spillway model tests with the ratio of design head to crest height H_d/z of 0.083 to 1.00. The approach depths were great enough to have negligible effects on the coefficient of discharge; and the tailwater was not high enough to cause submergence effects.

d. Submergence is defined as the ratio of the tailwater depth over a weir crest,to the head on the weir (N/H). A study was made /7/, /8/, and /9/ to determine the spillway discharge coefficient as a function of the submergence and the total head. The results of the study are shown on Plate 609. It is recommended that the $0.5H_d$ curve be used for all head ratios below $0.5H_d$.

e. Dimensionless Crest Profiles. The design discharge coefficient of a spillway /10/ may be determined by comparing its crest profile with a series of dimensionless crest profiles of known rating. This method integrates the other factors listed above, and is based on model studies and prototype tests of numerous dams The general procedure is to match a dimensionless profile of the spillway section with dimensionless profiles of structures with known coefficients of discharges as follows:

(1) Divide the coordinates of the spillway profile by the estimated design head determined as described in paragraph 98. The axis of coordinates is taken as the high point of the spillway crest.

(2) Plot the dimensionless crest profile ($x/_{Hd}$, $y/_{Hd}$) on transparent paper to a scale of 60 units per inch with the design head equal to 100 units.

(3) Superimpose the dimensionless profiles of step (2) on the Plates 610, 611, and 612 which are also plotted to a scale of 60 units per inch.

(4) Select the profiles most nearly matching the computed dimensionless profile and average their coefficients of discharge. In matching the crest profile for an overfall spillway, it is more important to match the upstream and downstream faces simultaneously rather than the coordinate axis.

f. Ratio of Actual Head to Design Head on Crest. After the coefficient of discharge for the design head has been determined, it is desirable to plot a head-discharge coefficient curve. Plate 613 gives the relationship of the coefficient of discharge /10/ for heads other than the design head on free overfall spillways. For any ratio of the actual head to the design head, a ratio of the discharge coefficient to the design discharge coefficient may be determined. Knowing the design coefficient of discharge, a head vs. discharge coefficient curve may be plotted for the complete range of heads desired.

103. Discharge Rating Curve. The discharge rating curve of a high ogee crested spillway would be determined as follows:

(1) Determine the design head, effective length of spillway crest, and the coefficient of discharge for the design head by the methods described in paragraphs 98, 99 and 102, respectively.

(2) Compute and plot the head vs. C_q curve by method given in paragraph 102.

(3) Assume various reservoir water surface elevations and determine the discharge coefficients from the curve in step (2) above.

(4) Compute the discharge for each reservoir elevation by equation 6-1.

(5) Plot the reservoir elevation against the resulting discharge to obtain the discharge rating curve.

104. Example. The computation of the discharge rating curve for the American Falls Dam with spillway gates fully raised is shown on Plate 614.

SECTION C: CHUTE SPILLWAYS

105. Definition. Chute spillways are a common type spillway designed for earth dams, and are generally isolated from the dam proper. This type of spillway is normally constructed in a saddle along the rim of the reservoir or near the abutment of the dam. The elements of a chute spillway consist of an approach channel, the weir or control section, and the chute or discharge channel. The control section of a chute spillway is normally constructed as an ogee weir with the overfall suppressed. Some chute spillways have the control section designed as broad-crested weirs. An example of a typical chute spillway is shown on Plate 615 which depicts the spillway plan and section of Kachess Dam.

106. Discharge Capacity of Ogee Weir. The discharge over an ogee weir type of chute spillway is computed by the basic weir equation

$$Q = C_q L H^{1.5} \qquad (6\text{-}1)$$

as described in par. 97.

107. Discharge Capacity of Broad-Crested Weir. The head-discharge relation over a broad-crested weir is computed by assuming a series of critical depths on the crest section of the weir. The corresponding head (H) upstream from the control section is equal to the sum of the velocity head and critical flow depth plus an additional proportion of the velocity head to allow for the approach losses. The following formulas are applicable:

$$Q_c = (gA^3/b_w)^{0.5} \qquad (6\text{-}6)$$

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$$h_v = A/2b_w \qquad (6\text{-}7)$$

$$H = y_c + (1{+}K)h_v \qquad (6\text{-}8)$$

where

Q_c = critical discharge in cfs at depth y_c
A = area of flow section in sq. ft. at depth y_c
b_w = top width of flow section in ft.
h_v = velocity head in ft.
y_c = critical flow depth in ft.
K = approach channel head loss coefficient normally assumed to be 0.1
H = total upstream head on crest in ft.

Reference is made to Par. 39 for discharge coefficients for broad-crested weirs.

108. Design Head. The design head is estimated in the same manner as for the spillways described in Par. 98.

109. Discharge Coefficient. a. The coefficient of discharge for the design head is determined as follows:

(1) Plot a dimensionless crest profile as $^x/H_d$ vs. $^y/H_d$ the same as for the spillway crest described in Par. 102.

(2) Superimpose the dimensionless crest profile on the profiles /10/ shown on Plated 616 and 617, and select the profiles that most nearly match the computed profile.

(3) Average the coefficients of discharge for the above selections to determine the coefficient of discharge at design head.

b. In matching the computed profile to the typical standard profiles given on the plates, it should be kept in mind that the flat portion of the chute immediately downstream of the crest section would have a more pronounced effect on the coefficient of discharge than the upstream approach depth. Therefore, it is more important to match the chute floor immediately downstream of the overfall than the upstream approach floor.

c. The head-discharge coefficient curve /10/ is computed in the same manner as for the high ogee spillway; and the ratios of C_q/C_d and H/H_d should be taken from Plate 618.

110. Approach Channel. Relatively long approach channels are frequently required for chute spillways because of their remote location from the dam. A dischaage rating curve computed for the spillway should consider the head loss from the reservoir to the spillway crest. The drop in water surface from the reservoir to the spillway crest would be the sum of the friction and eddy loss in the approach channel, and the drawdown of the water surface at the crest of the

weir. The head loss in the approach channel would be computed by a backwater computation as described in Par. 85. This friction head would be added to the head determined by the weir equation to give the total reservoir head. The total reservoir head or elevation is plotted against the discharge forming a discharge rating curve.

111. Tailwater. Submergence occurs at the weir of a chute spillway when the tailwater level is high enough to affect the discharge. If the bottom slope of the chute near the weir is less than critical, the discharge would normally be affected by tailwater conditions. If the bottom slope of the chute is greater than the critical slope, the discharge would be affected by the relative elevation of the chute bottom with respect to the weir crest elevation. The chute normally has a bottom slope greater than the critical slope. The control point for supercritical or rapid flow conditions is usually at the weir crest. The water surface profile for supercritical flow conditions would be computed in the downstream direction from the weir crest as discussed in Par. 77 and 85. To begin the flow line computation, the flow depth and the location of the control point on the weir crest is determined as follows:

(1) Compute the minimum specific energy head for the flow (equal to the critical depth plus the corresponding velocity head),

(2) Subtract the minimum specific energy head from the reservoir pool elevation to obtain the elevation of the spillway face at the control point. The water surface elevations for the succeeding sections downstream would be computed by the methods given in Par. 85. For purposes of determining the amount of submergence of the weir, it is only necessary to determine the tailwater profile in the chute a short distance below the control section. For chute spillways with ogee crests /11/, the upper and lower nappe profiles for any head are determined as discussed in Par. 26. The nappe thickness or depth of flow downstream from an ogee spillway crest is estimated as the difference between the nappe profiles at any point on the spillway face. The following table is useful in estimating the depth of flow on the spillway face. Both the nappe thickness, and the vertical coordinate of the spillway face, are expressed as ratios of the head on the spillway crest. The origin of coordinates is the crest of the spillway.

Vertical Coordinate of the Spillway Face Downstream from the Crest ($^y/H$)	Theoretical Nappe Thickness ($^t/H$)
1.0	0.368
1.6	0.317
2.0	0.294

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112. Submergence. The effect of submergence on a low-head dam /11/, which is normal design for a chute spillway, is shown as a series of dimensionless curves on Plate 619. The curves show the effects of submergence and also the influence of the apron floor on the discharge coefficient. The dashed lines, designating constant decrease in the discharge coefficient, indicates the effect of the downstream floor. When the dashed lines are horizontal, the decrease in the coefficient is due to submergence. As the apron floor approaches the crest of the spillway, the decrease in the discharge coefficient approaches 23 percent, and the dam becomes virtually a broad-crested weir. For values of $(h_d + y)/H$ from 1.0 to 1.7, the decrease in the discharge coefficient is caused by a combination of apron effect and submergence.

113. Crest Piers and Pier Contraction Coefficients. The effect of piers with the resulting contraction on the flow over a spillway would be computed by methods given in Par. 99.

114. Discharge Rating Curve. a. The discharge rating curve of a chute spillway is determined as follows:

(1) Determine the design head, effective length of spillway crest, and the coefficient of discharge at the design head by the methods described in Pars. 98, 99, and 109, respectively.

(2) Compute and plot the head vs. discharge coefficient curve by the methods described in Par. 109.

(3) Assume a reservoir water surface elevation.

(4) Assume no head loss, as a first approximation, in the approach channel and determine the discharge coefficient from the curve in step (2) above.

(5) Compute the discharge for the assumed reservoir elevation by equation 6-1.

(6) Compute the head loss in the approach channel for the computed discharge of step (5) by the methods described in Par. 110.

(7) Recompute the discharge coefficient and the discharge based on the new head and adjust by trial until a reasonable balance is secured.

(8) Assume other reservoir elevations and repeat steps (3) to (7).

(9) Plot the reservoir water surface elevation against the discharge for the rating curve.

b. Plate 618, showing the relation of the ratios of C_q/C_d and H/H_d, has integrated the effects of submergence by the tailwater and, therefore, it was not considered in the above procedure.

115. Example. The computation of the discharge rating curve for the Kachess Dam spillway with the tainter gates fully open is shown on Plate 620.

SECTION D: SIDE CHANNEL SPILLWAYS

116. Definition. Side channel spillways, or lateral flow spillways, are a composite type spillway consisting of an overflow weir, usually lying at right angles to the axis of the dam, and discharging into a narrow channel in which the direction of flow is approximately parallel to the weir crest. An example of a side channel spillway is shown on Plate 621. When the overflow weir is level and not submerged, the water entering the channel is assumed to be constant per foot of length of weir and the quantity of flow in the channel increases at a constant rate progressively downstream. It is assumed that all the energy of the water flowing over the spillway crest is dissipated in turbulence and the flow down the spillway channel is caused entirely by the slope of the water surface in the channel.

117. Discharge Capacity. The discharge capacity of a side channel spillway is computed by the basic weir equation described in paragraph 97. At heads near the design head, the tailwater often will partially submerge the upper portion of the weir crest and reduce the discharge. The head, effective crest length, and coefficient of discharge would be computed by the methods described for ogee weirs.

118. Water Surface Profile. a. The water surface profile /12/ in the discharge channel or the outlet conduit of a side channel spillway normally is the control at the design head. The following procedure is used to determine the surface curve in the channel.

b. On Plate 622 consider the conditions of flow in the section of channel between A and B. If (q) is the discharge per foot along the crest of the weir, and (L) is the length of the weir; then $qL = Q$, the total discharge. For any two sections (ΔL) apart, the discharge is $(q\Delta L)$. Let Q_1 = discharge at section 1-1, then $Q_1 + q\Delta L = Q_2$, the discharge at section 2-2. The velocity in the channel at section 1-1 is V_1 and at section 2-2 it is $V_1 + V = V_2$. The drop, or rise, in the water-surface curve Δy in the reach of length ΔL is expressed by the formulas:

$$\Delta y = \frac{Q_1 (V_1 + V_2)}{g (Q_1 + Q_2)} \left(\Delta V + \frac{qV_2 \Delta L}{Q_1} \right) \quad (6\text{-}9)$$

$$\Delta y = \frac{Q_2 (V_1 + V_2)}{g (Q_1 + Q_2)} \left(\Delta V + \frac{qV_1 \Delta L}{Q_2} \right) \quad (6\text{-}10)$$

c. When (Q_1) and (V_1) or (Q_2) and (V_2) are known, select a trial value for (Δy), compute the corresponding depth, and solve for (Q_2) and (V_2) or (Q_1) and (V_1). Compute friction losses as explained in paragraph 80. If the sum of the depth at the lower point plus the trial (Δy) plus the friction between the points is equal to the sum of the depth at the upper point plus the rise in bottom elevation then the trial (Δy) is correct. If found to be incorrect, a new trial (Δy) is selected and the solution is repeated until a check is obtained.

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d. In order to apply equations 6-9 and 6-10 it is necessary to start at a point in the channel where the velocity, area, and depth are known. These are known only at the central section, or sections, where the flow is at critical depth.

e. The control section may occur either below the end of the weir, where the discharge is constant, or within the channel section above the end of the weir, where the discharge is variable. If the slope of the channel above the end of the weir at "B" (see Plate 622) is not sufficient to support flow at critical depth and if the slope of the channel below "B" is sufficient to support flow at less than critical depth, the control occurs at "B". If the control section is at a point below "B", the water surface curve is computed up to section "B" by backwater computations as described in paragraph 85.

f. The control section, when occurring in the channel above "B", is located as follows: Compute critical velocities and depths for several sections of the channel for the quantity of discharge at these sections. Starting at the upstream end of the channel compute the drop in water surface necessary to produce flow at critical depth, using equation 6-9. Using any convenient scale and starting at an arbitrary elevation, plot the theoretical water surface curve. Lay off a theoretical bottom profile from the theoretical water surface curve with the computed depths. These two profiles will represent the relation between the water surface elevation and the bottom profile of the channel having the same cross sections as the one under consideration in which critical flow would occur throughout its length. Plot the profile of the actual channel bottom to the same scale.

g. The control section is at that point in the channel where the tangent to the profile of the theoretical bottom and the tangent to the profile of the actual bottom have the same inclination. In sections of the channel where the theoretical bottom profile is flatter than the actual bottom profile, the flow will be less than the critical depth; and in sections where the theoretical bottom profile is steeper than the actual bottom profile, the flow depth will be greater than the critical depth.

h. The water surface profile upstream from the control section is computed by equation 6-9. The starting elevation at the control section is the critical depth. The water surface profile downstream from the control section is computed by equation 6-10.

119. Submergence. Submergence of a portion of the weir by the channel backwater will reduce the discharge over the weir along the section submerged. The amount of submergence which may occur before resulting in an appreciable change in weir discharge has been discussed in paragraphs 102 and 112. For 80 percent submergence, weir discharges will be decreased by approximately 10 percent of the free discharge.

120. Swelling. A safe value for the percentage of swell resulting from the air entrapped by the water is a water surface at five percent greater depth than computed.

121. Discharge Rating Curve. The discharge rating curve would be computed in the same general manner as described in Sections A and B, Chapter VI, and as shown on Plate 614. Discharges approaching the design head are checked by computing the water surface profile in the outlet channel as described in paragraph 118. If the tailwater submerges the weir crest the discharge is modified in accordance with the method described in paragraph 102.

122. Example. The computation of a water surface profile up the rectangular channel of a side channel spillway is given on Plate 622. The total discharge over the weir is assumed to be 4000 cubic feet per second.

SECTION E: VERTICAL SHAFT SPILLWAY

123. Definition. A vertical shaft spillway, also called a "morning glory" or "glory hole" spillway, consists of a circular funnel shaped intake converging into a vertical shaft that bends into a nearly horizontal conduit which normally discharges into a stilling basin. The funnel shaped intake section normally has one of two profiles, an ogee-crest weir profile, or a broad-crested weir profile. A typical ogee and broad crested morning glory spillway are shown on Plate 623. The ogee-crested weir /13/, as shown on Plate 624, consists of a free-fall section, a decreasing shaft or transition section, a vertical shaft section and a horizontal conduit section. The free-fall section has the profile of the lower nappe of a sharp-crested weir. The transition section is usually circular in cross-section of decreasing radius to provide a smooth transition from the free-fall section to the constant diameter vertical shaft and outlet conduit. The broad-crested weir, as shown on Plate 624, consists of a nearly flat weir section, a free-fall section, transition section, and vertical shaft and outlet section. The weir section is given sufficient length to allow the approach flow to drop to the critical depth with adequate slope to balance the effects of friction and the convergence of the flow filaments. The other elements of the vertical shaft spillway with a broad-crested weir, are the same as those of the ogee-crest morning glory spillway. The advantages of a vertical shaft spillway are the small space it occupies and the relatively small head required to develop the design discharge. A disadvantage of this type of spillway is that its capacity does not materially increase when the head on the spillway increases above the design head.

124. Discharge Capacity. The discharge capacity of a vertical shaft spillway is computed by either a weir equation or a pipe flow equation. Discharges below the design discharge are normally controlled by the weir capacity and would be computed by the basic weir equation. At higher discharges, the capacity of the spillway is controlled by the capacity of the shaft and outlet conduit. The discharge under this condition is computed by the basic pipe flow formula.

125. Effective Crest Length and Design Head. The design head and effective crest length of a morning glory spillway would be computed in the same manner as for an ogee spillway (see Pars. 98 and 99).

126. Coefficient of Discharge. The weir coefficient of discharge /14/ at the design head of an ogee-crested shaft spillway is a function of the various factors discussed for ogee weirs in Section B, Chapter VI, and the radius of curvature of the weir, as shown on Plate 625. It is to be noted from Plate 624 that the design head is measured from the crest of the weir and the radius is measured to the weir crest. The coefficient of discharge of a broad-crested weir is a function of the shape of the weir. Typical values of several types of broad-crested weirs are given in paragraph 39.

127. Discharge Rating Curve. a. The discharge rating curve of a vertical shaft spillway is determined from parts of two rating curves. The lower segment of the rating curve is controlled by the capacity of the weir until it is submerged by back pressure from the outlet conduit. The second segment of the rating curve is controlled by the capacity of the shaft and outlet conduit. The discharge rating curve turns sharply upward near the design head so that very little increase in discharge occurs for additional head on the spillway.

b. Weir Rating Curve. The lower portion of the discharge rating curve of a shaft spillway would be computed by the basic weir formula

$$Q = C_q L H^{1.5} \qquad (6\text{-}1)$$

where

Q = discharge in cfs
L = the effective length of crest, computed as the crest perimeter of the intake funnel.
H = upstream head measured from the crest of the weir.
C_q = coefficient of discharge.

The computation procedure would be the same as described in paragraph 103.

(1) Ogee Weir. The coefficient of discharge for an ogee-crested morning glory spillway would be determined in the same manner as for a straight ogee spillway and then modified for the convergence of the stream filaments by use of Plate 625. The spillway profile of a morning glory spillway is normally shaped to fit the lower nappe of a sharp crested weir at the design head. The design head would be determined by the methods described in Par. 98 and from Plate 601. The discharge coefficient at the design head is 3.97. The design head discharge coefficient is reduced by an amount determined from Plate 625 due to the convergence of the flow filaments into the spillway funnel. The coefficient of discharge is also modified for

heads other than the design head by use of Plate 613. The coefficient of discharge of an ogee spillway is reduced about 20 percent when the head is reduced to about 25 percent of the design head, and when the ratio of design head to the radius is about 0.5.

(2) Broad-Crested Weir. The discharge from a vertical shaft spillway with a broad-crested weir would be computed by the basic weir equation with a coefficient of 3.0 and the head (H) measured from the flat-crested weir (see Plate 624). The value of C_q would be lowered if unfavorable flow conditions prevailed in the approach channel.

c. Shaft Rating Curves. The upper portion of the discharge rating curve of a vertical shaft spillway is computed as a conduit flowing full. The total energy head is consumed in the outlet velocity head plus the head loss due to friction and bends. The method of computing the discharge for a given head or the head for a given discharge is discussed in paragraph 59. The head loss due to bends in large conduits would be computed by the following equation:

$$K_b = \frac{1}{\pi(\log_e r/D + \pi/2)} \qquad (6\text{-}11)$$

where

r = radius of bend
D = diameter of conduit

Head loss due to bends in large conduits is discussed further in paragraph 166. The value of K_b for various values of r/D is given on Plate 802.

128. Example. The computations of the discharge rating curves for an ogee-crested shaft spillway and a broad-crested shaft spillway are shown on Plates 626 and 627 respectively.

SECTION F: SIPHON SPILLWAYS

129. Definition. A siphon is defined as a tube bent to form two legs of unequal length, by which a liquid can be transferred to a lower level, over an intermediate elevation, by atmospheric pressure forcing the liquid up the shorter branch of the pipe immersed in it, while the excess of weight of the liquid in the lower branch causes a flow. The siphon spillway is a siphon device used to discharge water over an intermediate elevation and regulate pool levels within relatively close limits. The component parts of a siphon spillway are the inlet, the upper leg, the throat, the lower leg and the outlet. Examples of typical siphon spillways are shown on Plate 628. The advantage of a siphon spillway is that it attains full discharge capacity upon priming, without having tobuild up head as for an overflow weir, also the priming head is only several tenths of a foot and therefore the spillway gives close pool regulation. The disadvantage

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of this type of spillway is that the capacity does not materially increase for increased heads above the priming or design head.

130. Maximim Siphonic Head. The absolute maximum siphonic head on a siphon spillway would be equal to 34 feet. This is equivalent to the height a column of water will rise in a vacuum due to an atmospheric pressure of 14.7 pounds per square inch at sea level, and at a temperature of 32°F. The maximum effective head on a siphon is considerably less than the absolute maximum of 34 feet. As the absolute pressure approaches zero, air is released from the water and accumulates on the throat of the siphon causing a reduction in flow area and discharge. Very low absolute pressures may cause sufficient air to collect at the throat to interrupt the flow intermittently causing vibration and cavitation. The outlet of a siphon must be flowing full or be submerged to have full siphonic action. If the siphon does not flow full the siphonic head would be measured from the throat crest to the point of separation in the outlet leg. European authorities /15/ suggest that the hydraulic gradient should be at least 2 meters below the absolute hydraulic gradient. American design practices /16/ indicate a negative throat pressure equivalent to 24 feet of water to be the practical design limit. Siphons have been designed for heads of above 30 feet.

131. Throat. The throat of a siphon spillway is generally of rectangular section curved to circular arcs. The maximim velocity is at the crest (bottom) of the throat, and the product of velocity times radius is constant across the flow section. The velocity depends upon the degree of rarification at the throat. A vacuum is not produced even under the most favorable conditions and thus the maximum effective head may be about 24 to 30 feet at sea level.

132. Discharge Capacity. a. Siphons attain full discharge immediately upon priming. Since the maximum head available in the production of velocity, neglecting losses, is approximately 24 to 30 feet, any head in excess of this, minus the head required to overcome losses, is not effective in producing discharge and may be disregarded. The maximum velocity head at the throat crest (h_{vc}), will be the velocity head due to the atmospheric (absolute) pressure head (h_{at}) minus the structure losses between the inlet and the crest ($\Sigma K_s h_v$), plus the head (h) due to the superelevation of reservoir level over the crest. The maximim head which can be effective in producing flow with a full conduit is then

$$h_{vc}(\max) = h_{at} + h - h_{pc} - \Sigma K_s \, h_v \qquad (6\text{-}12)$$

where the pressure head at the crest of the throat (h_{pc}) is zero. Siphons are usually designed so that h_{pc} will not be less than 10 feet.

b. Flow in the circular curved throat section closely follows free vortex action and the product of velocity times radius is constant.

The maximum unit discharge (q) for the throat is:

$$q = V_{cr} \, r_c \, \log_e(r_s/r_c) \qquad (6\text{-}13)$$

where

V_{cr} = velocity at crest of throat, maximum throat velocity
r_c = radius of crest of throat
r_s = radius of summit of throat

A short table for various values of $\log_e$ is found on Plate 629. In equation 6-13:

$$V_{cr} = (2gh_{vc})^{0.5} \qquad (6\text{-}14)$$

The total discharge

$$Q = q \text{ times throat width} \qquad (6\text{-}15)$$

c. The discharge capacity of siphons may be obtained by equating the gross head, from water surface to water surface or outlet, (H), to the sum of the structure head losses and the velocity head loss at the outlet. With the head losses in terms of velocity head in the throat section, the following equations are employed:

$$H = (\Sigma K_s + K_o)\frac{V^2}{2g} \qquad (6\text{-}16)$$

$$V = \left(\frac{2gH}{\Sigma K_s + K_o}\right)^{0.5} \qquad (6\text{-}17)$$

$$q = V(\Sigma K_s + K_o) \qquad (6\text{-}18)$$

$$Q = A_t\left(\frac{2gH}{\Sigma K_s + K_o}\right)^{0.5} \qquad (6\text{-}19)$$

where

q = discharge per unit width
Q = discharge
K_s = sum of structure head loss coefficients
K_o = velocity head loss coefficient at outlet
V = mean velocity in throat section
H = total head
A_t = area of throat

The relation of mean throat velocity to maximum throat velocity /5/ is found on Plate 630.

d. Within the limits of permissible velocity, siphon discharge may be represented by:

$$Q = C_q A_t (2gH)^{0.5} \qquad (6\text{-}20)$$

C_q usually varies from 0.6 to 0.8

e. H is the difference between the water surface in the reservoir and the tailwater provided it is not greater than the limiting head of 34 feet. H in equation 620 is always less than 34 feet.

f. The limiting head decreases with increasing altitude as shown by Plate 629. The decrease in atmospheric pressure may be taken as 0.5 lb. for each 1000 feet above sea level, hence the siphonic head decreases, approximately, 1 foot for each 850 feet of altitude.

g. A severe storm may cause a drop of as much as two feet of head.

133. Structure Head Losses. The loss of head due to form and friction losses is described in Chapter 4 and are applicable to siphon spillways. The bend loss at the throat of a siphon is computed as follows:

$$h_b = K_b \frac{v^2}{2g} \qquad (6\text{-}21)$$

$$K_b = 0.23\ (r_s - r_c)/r_c \qquad (6\text{-}22)$$

where

h_b = bend head loss
r_c = radius of crest of throat
r_s = radius of summit of throat

134. Discharge Rating Curve. The discharge rating curve of a siphon spillway consists, for all practical purposes, of a near vertical line. The capacity of the siphon upon priming is practically constant for all reservoir elevations.

135. Example. The discharge capacities of the siphons shown on Plate 628 are computed as examples and shown on Plates 631, 632, and 633.

136. References.

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/15/ Rock, Elmer. "Design of High Head Siphon Spillway". <u>Trans. A.S.C.E.</u>, Vol. 105, 1940, p 1050.

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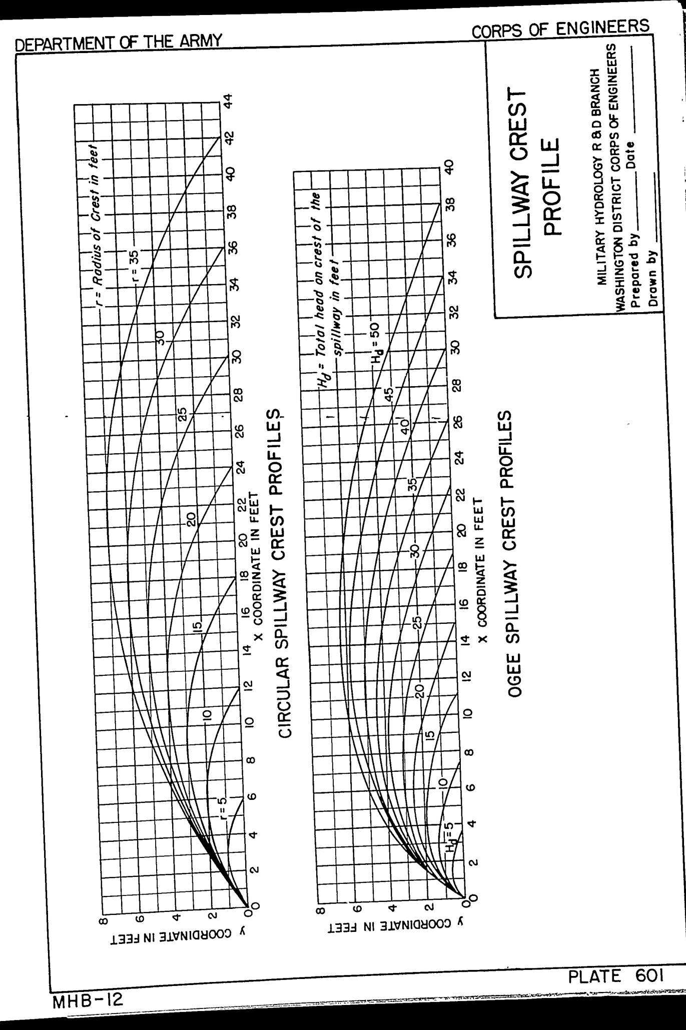
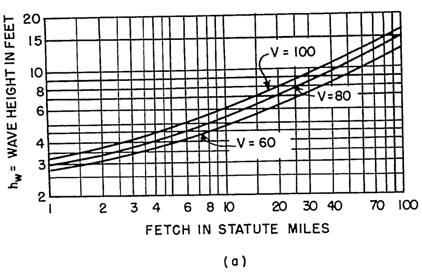
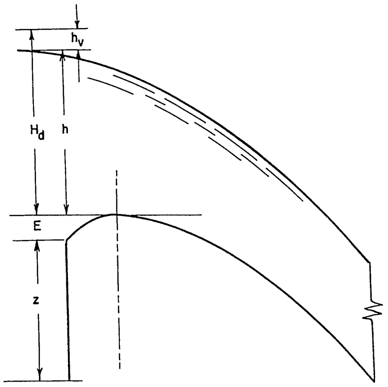




(a)



(b)

h = head on Weir

H_d = Design Head

h_v = Velocity head

E = Crest rise

z + E = Spillway Crest height

WAVE HEIGHT and WAVE NOTATION

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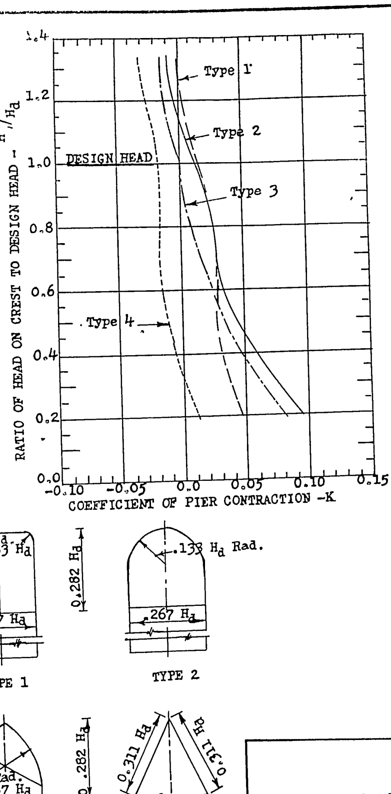

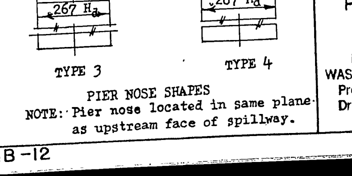

PIER NOSE SHAPES

NOTE: Pier nose located in same plane as upstream face of spillway.

PIER CONTRACTION COEFFICIENTS EFFECT OF NOSE SHAPE

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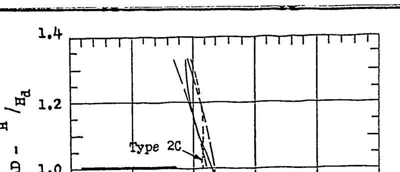

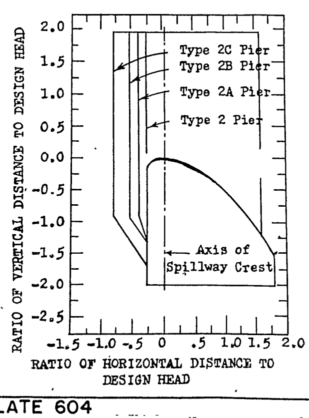

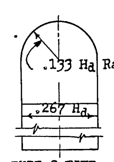

TYPE 2 PIER

PIER CONTRACTION COEFFICIENTS
EFFECT OF PIER LENGTH

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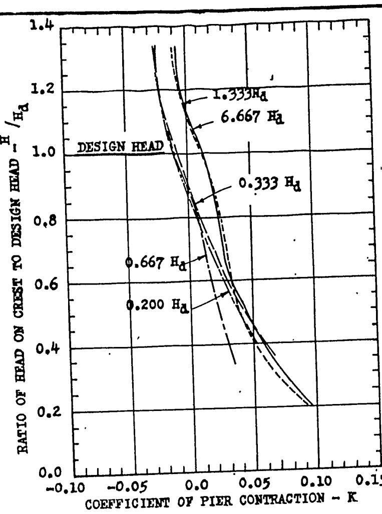

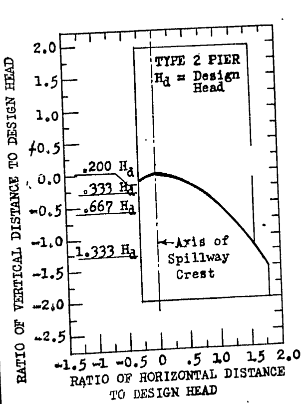

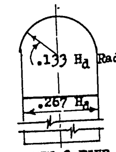

TYPE 2 PIER

PIER CONTRACTION COEFFICIENTS
EFFECT OF APPROACH DEPTH

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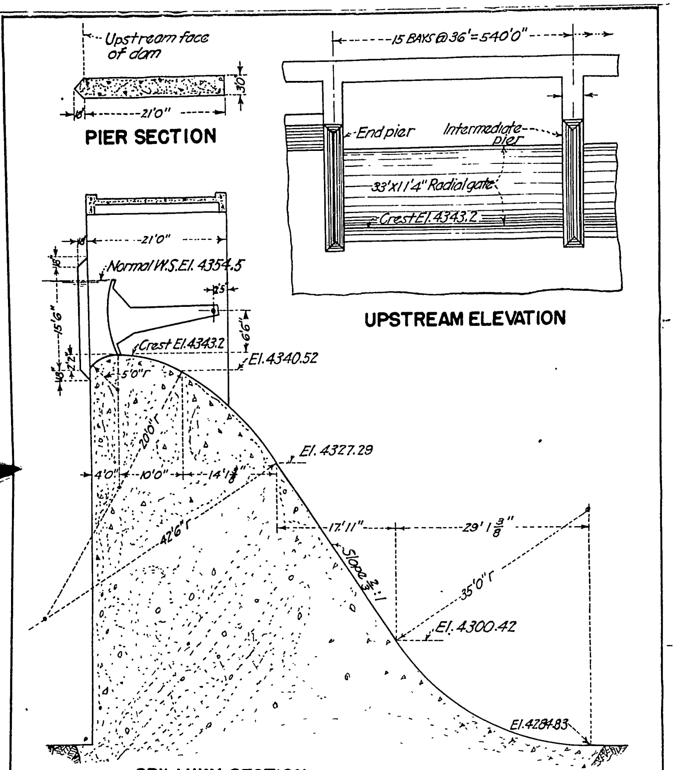

TYPICAL OGEE SPILLWAY

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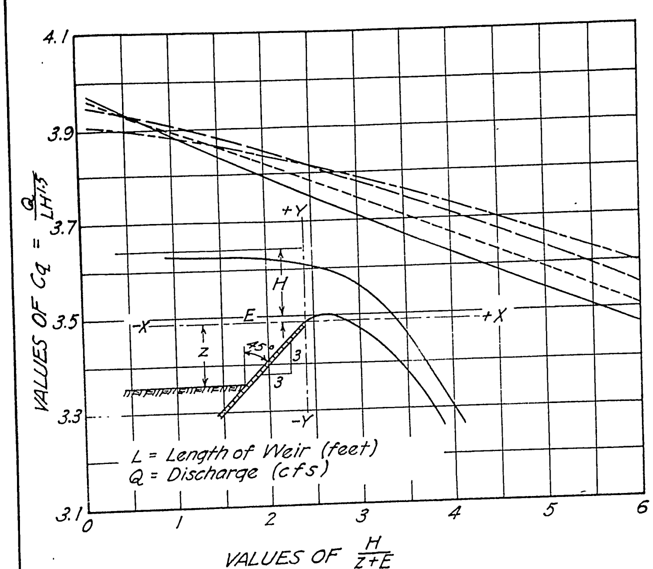
PLATE 606

MHB-12



LEGEND

WEIR INCLINED	ANGLE WITH VERTICAL
Downstream on 1:3 Slope	18.43°
" " 2:3 "	33.69°
" " 3:3 "	45.0
Vertical Face Weir Slope	

OGEE SPILLWAY DISCHARGE DATA

DISCHARGE COEFFICIENT vs. APPROACH DEPTH

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PLATE 607

MHB-12

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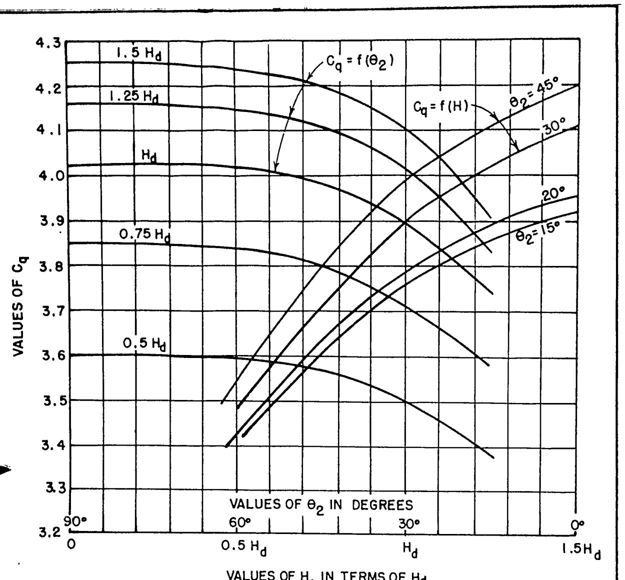
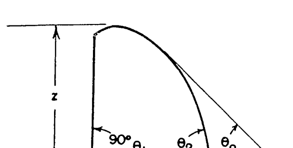


$$Q = C_q L H^{\frac{3}{2}}$$



WEIR PROFILE

OGEE SPILLWAY DISCHARGE DATA

DISCHARGE COEFFICIENT vs. ANGLE OF DOWNSTREAM FACE

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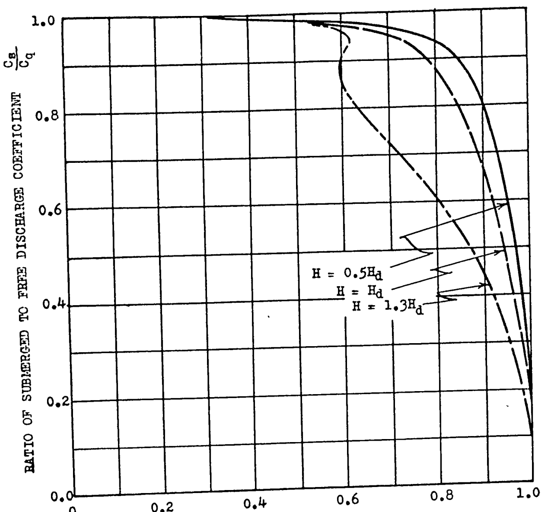

SUBMERGENCE $\left(\text{RATIO } \frac{N}{H}\right)$

NOTES:

H = Head on Weir
N = Depth of Submergence
H_d = Design Head for Crest
C_q = Discharge Coefficient, Free Flow
C_s = Discharge Coefficient, Submerged Weir

OGEE SPILLWAY DISCHARGE DATA, DISCHARGE COEFFICIENT VS. SUBMERGENCE

MILITARY HYDROLOGY R & D BRANCH
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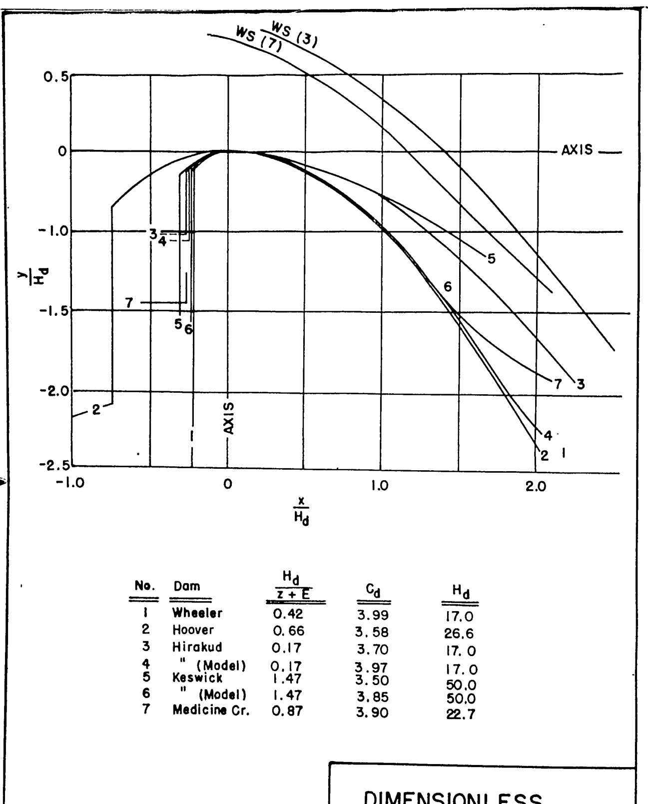

No.	Dam	$\frac{H_d}{z+E}$	C_d	H_d
1	Wheeler	0.42	3.99	17.0
2	Hoover	0.66	3.58	26.6
3	Hirakud	0.17	3.70	17.0
4	" (Model)	0.17	3.97	17.0
5	Keswick	1.47	3.50	50.0
6	" (Model)	1.47	3.85	50.0
7	Medicine Cr.	0.87	3.90	22.7

DIMENSIONLESS SPILLWAY PROFILES

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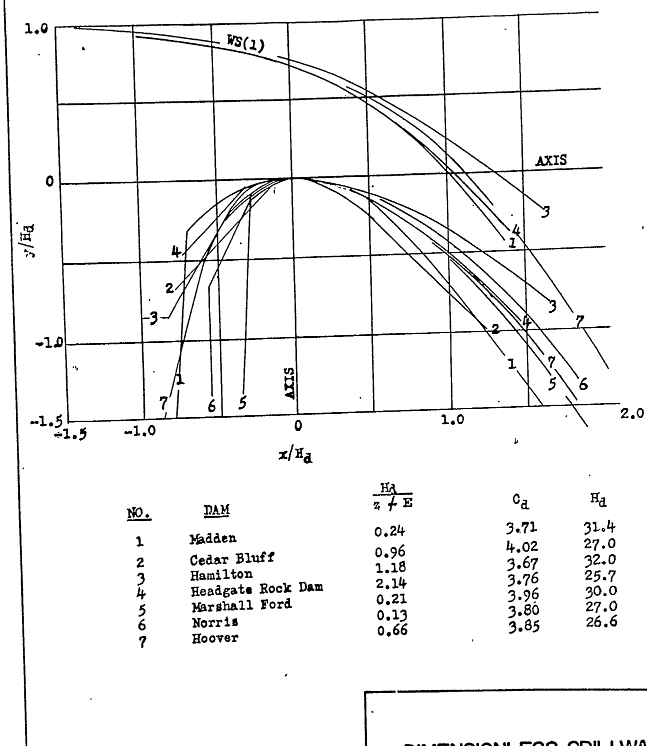
PLATE 610

MHB-12



NO.	DAM	$\frac{H_d}{z+E}$	C_d	H_d
1	Madden	0.24	3.71	31.4
2	Cedar Bluff	0.96	4.02	27.0
3	Hamilton	1.18	3.67	32.0
4	Headgate Rock Dam	2.14	3.76	25.7
5	Marshall Ford	0.21	3.96	30.0
6	Norris	0.13	3.80	27.0
7	Hoover	0.66	3.85	26.6

DIMENSIONLESS SPILLWAY PROFILES

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PLATE 611

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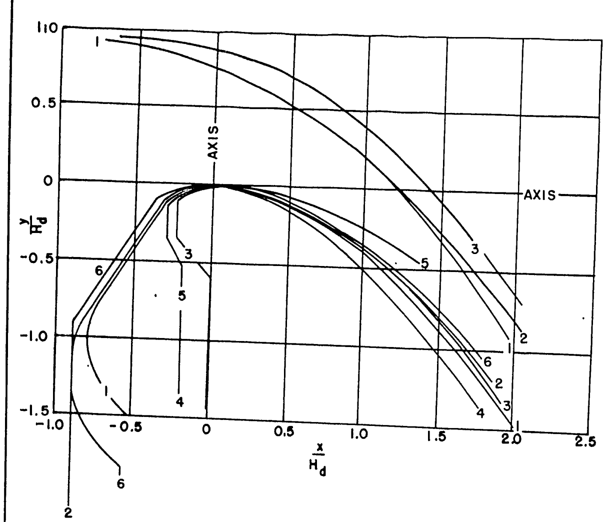

No.	DAM	$\frac{H_d}{z + E}$	C_d	H_d
1	Grand Coulee	0.09	3.86	31.6
2	Bhakra	0.09	3.68	28.0
3	Angostura	0.45	3.88	41.7
4	Wilson	0.27	3.98	20.5
5	Davis	0.55	3.59	50.0
6	Friant	0.08	3.65	19.0

DIMENSIONLESS SPILLWAY PROFILES

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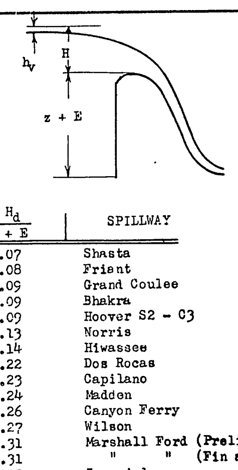

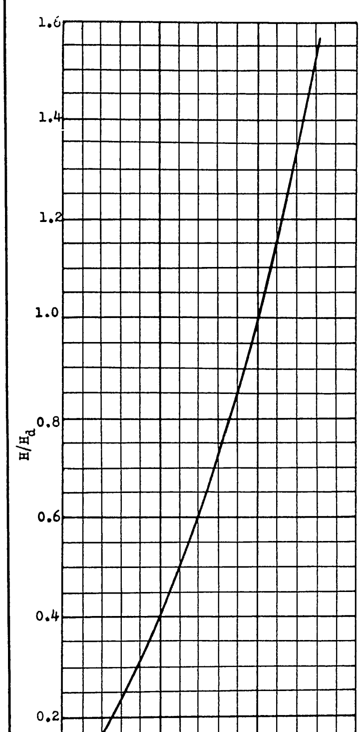

COEFFICIENTS OF DISCHARGE FOR OTHER THAN THE DESIGN HEAD (FREE OVERFALL SPILLWAYS)

$\frac{H_d}{z + E}$	SPILLWAY
0.07	Shasta
.08	Friant
.09	Grand Coulee
.09	Bhakra
.09	Hoover S2 - C3
.13	Norris
.14	Hiwassee
.22	Dos Rocas
.23	Capilano
.24	Madden
.26	Canyon Ferry
.27	Wilson
.31	Marshall Ford (Prelim.)
.31	" " (Final)
.37	Imperial
.42	Wheeler
.45	Angostura
.65	Moon Lake
.65	Davis (Prelim.)
.65	" (Final)
.67	Hoover S7 - C8
.67	" S3 - M1
.67	" S8 - C8
.67	" S4 - M3
.87	Medicine Creek (High)
.96	Cedar Bluff
1.20	Hamilton
1.50	Keswick
3.20	Trenton

OVERFALL SPILLWAY DISCHARGE COEFFICIENT vs. HEAD

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DETERMINATION OF THE DISCHARGE RATING CURVE FOR THE AMERICAN FALLS DAM

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-
(3) The basic data was taken from Plate 606.

DIMENSIONLESS CREST PROFILE

(4) The prototype crest dimensions were taken from Plate 606. Each dimension was divided by the design head (11.3 ft.) and tabulated in Col. 2, Table I.

(5) The dimensionless ratios of Col. 2, Table I, were plotted on transparent paper, to a scale of 60 units per inch with the design head equal to 100 units, and the profile drawn as shown.

(6) The dimensionless crest profile was superimposed over the profiles shown on Plates 610, 611, and 612. The transparency was matched with the upstream and downstream faces of the profiles simultaneously, not the co-ordinate axis. The American Falls profile nearly matched the Keswick Dam and Davis Dam profiles with C_d = 3.50 and 3.59, respectively. Spillways with straight vertical offsets in the upstream face perform very much the same as a vertical face spillway. From the above comparison, C_d was estimated to be 3.55 for the American Falls Dam.

DISCHARGE RATING CURVE

(7) The discharge rating curve of the American Falls Dam was determined as described in the following steps:

1. The head ratios were assumed as shown in Col. 1, Table II.

2. The heads on the spillway (Col. 2) were computed as the product of the design head and the ratios of Col. 1.

3. The reservoir water surface elevations (Col. 3) were determined as the sum of the spillway crest elevation and the heads of Col. 2.

4. For each ratio of actual head to design head in Col. 1, the ratio of discharge coefficient to design head discharge coefficient was determined from Plate 613, and tabulated in Col. 4.

5. The discharge coefficient (Col. 5) was determined as the product of the design head discharge coefficient and the ratios of Col. 4.

6. The discharge over the spillway (Col. 6) was computed by Eq. 6-1, in which the discharge coefficient was taken from Col. 5, the heads from Col. 2, and the effective length of spillway was computed by Eq. 6-5.

   The effective length of spillway crest was computed as follows:

   $$L = L' - KNH$$

   L' = 15 bays x 33 ft. = 495 ft.

   K = Pier coefficient taken from Plates 603 and 604 with the head ratios = 0.2 and 1.2, respectively.

   N = 2 contractions x 15 bays = 30 contractions.

   L = 491.6 ft. & 490.9 ft. A constant crest length of 491 ft. was used for the conplete range of heads without appreciable error.

(8) The discharge rating curve was plotted from values in Cols. 3 and 6 of Table II.

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DETERMINATION OF THE DISCHARGE RATING CURVE FOR THE AMERICAN FALLS DAM

COMPUTATION SHEET

Item

INITIAL DATA

(1) Profile of the spillway crest of the American Falls Dam. Crest elevation = 4343.2 feet msl.

(2) The design head = 11.3 feet.

(3) 15 spillway gates each with 33 feet clear opening.

(4)

TABLE I
DIMENSIONLESS CREST PPROFILE

Length or distance Col. 1	Length/H_d Col.2
Upstream face from crest	0.354
Upstream crest radius	0.442
Upper segment of downstream crest radius	1.770
Lower segment of downstream crest radius	3.770
Upper P.T. of downstream crest	0.237
Lower P.T. of downstream crest	1.408

(5) The dimensionless crest profile of the American Falls Dam is shown at the right.

(6) Coefficient of discharge at the design head = 3.55.

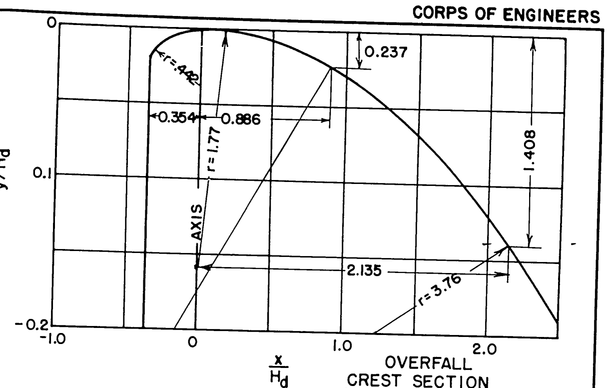
(7)

TABLE II
DISCHARGE RATING CURVE COMPUTATIONS

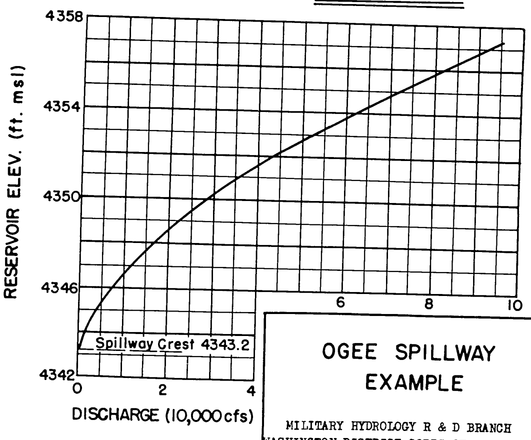
H/H_d (a) Col. 1	H (ft.) Col. 2	Reservoir W.S. Elev.(ft.msl.) (b) Col.3	C_q/C_d (c) Col.4	C_q Col.5	$Q = C_qLH^{3/2}$ (cfs) (d) Col.6
0.0	0	4343.20	-	-	0
0.2	2.26	4345.46	0.85	3.01	5020
0.4	4.52	4347.72	0.90	3.19	15000
0.6	6.79	4349.99	0.94	3.33	28100
0.8	9.05	4352.25	0.975	3.46	46300
1.0	11.30	4354.50	1.00	3.55	66200
1.2	13.57	4356.77	1.025	3.64	89500

(a) H_d = 11.30 feet.
(b) Spillway crest elevation = 4343.20 feet, msl.
(c) C_d = 3.55
(d) L = 491 feet.

(8) The discharge rating curve was plotted from values in Cols. 3 and Col. 6 of Table II.



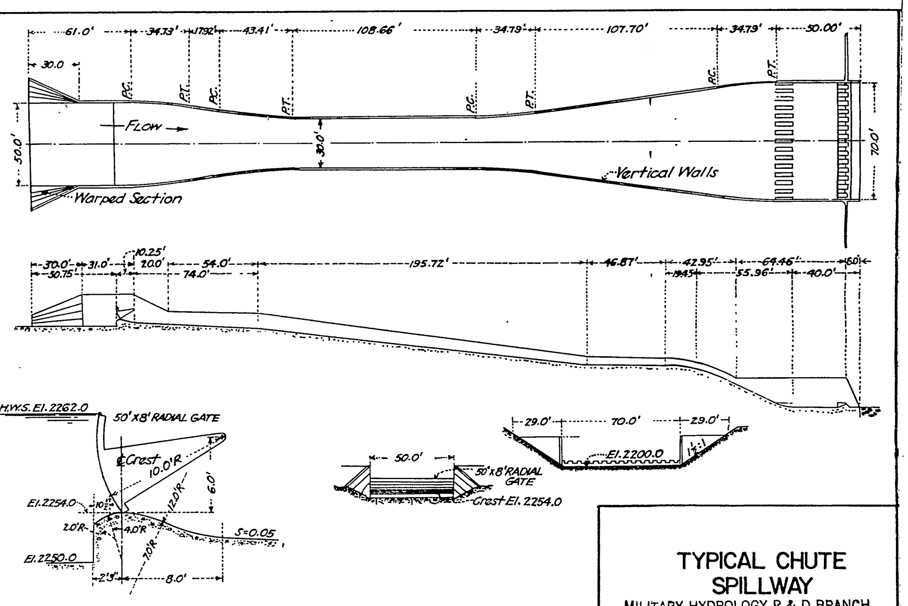
OVERFALL CREST SECTION



OGEE SPILLWAY EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by________ Date ________
Drawn by ________

PLATE 614B



TYPICAL CHUTE SPILLWAY

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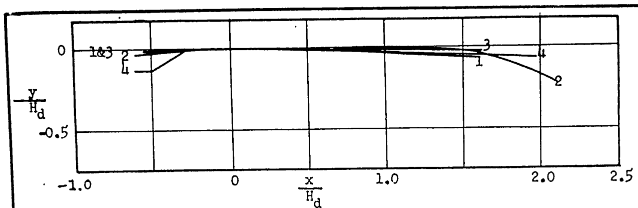
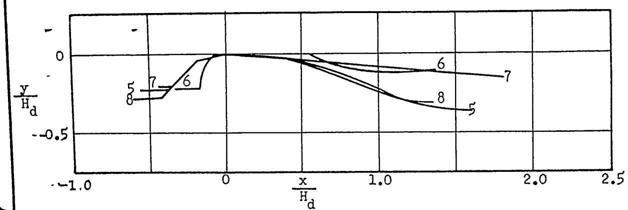
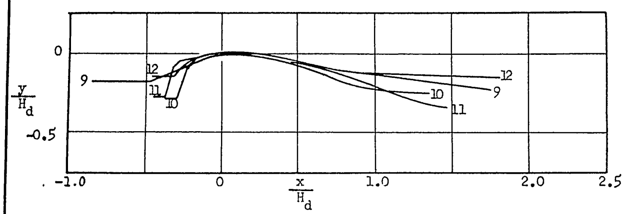


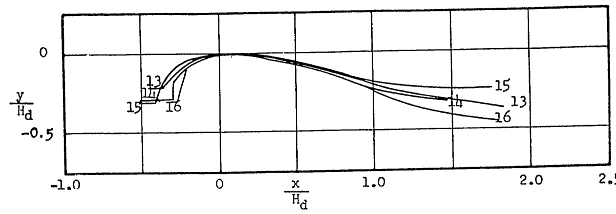


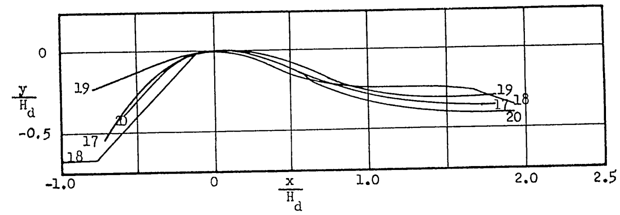


NO.	NAME	H_d	z ≠ E	C_d
1	Agency Valley Dam	17.0	1.0	2.73
2	Pine View Dam	16.0	4.0	2.74
3	Rye Patch Dam	17.0	1.0	2.81
4	Alcova Dam	40.0	5.0	2.85
5	Alamogordo Dam	26.0	5.0	3.18
6	Horseshoe Dam	38.0	8.0	3.20
7	Green Mountain Dam	22.0	4.0	3.21
8	Moon Lake Dam	16.0	4.0	3.28
9	Shadow Mountain Dam	19.0	3.5	3.30
10	Falcon Dam	57.5	16.7	3.33
11	Boysen Dam	52.0	10.0	3.37
12	Cascade Dam	20.0	3.0	3.38

DIMENSIONLESS SPILLWAY PROFILES

MILITARY HYDROLOGY R&D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by_______ Date_______
Drawn by_______





NO.	NAME	H_d	z ≠ E	C_d
13	Anderson Ranch Dam	24.0	5.0	3.40
14	Vallecito Dam	19.0	5.0	3.42
15	Bartlett Dam	50.0	14.0	3.48
16	Tiber Dam	34.0	10.0	3.49
17	Boca Dam	15.4	5.0	3.50
18	Fresno Dam	16.0	11.5	3.52
19	Medicine Creek Dam	32.6	7.2	3.54
20	Bull Lake Dam	11.0	4.0	3.58

~DIMENSIONLESS SPILLWAY PROFILES~

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WASHINGTON DISTRICT CORPS OF ENGINEERS
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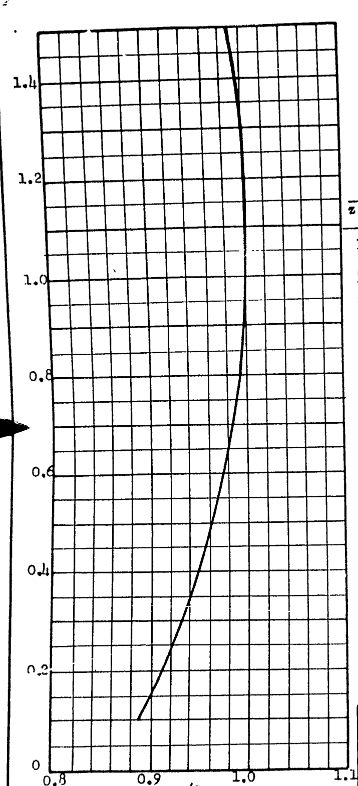
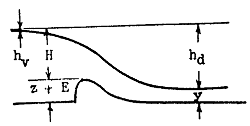


COEFFICIENTS OF DISCHARGE FOR OTHER THAN THE DESIGN HEAD (SPILLWAYS WITH OVERFALL SUPRESSED).



$\frac{H_d}{z + E}$	SPILLWAY	$\frac{h_d + y \text{ (Actual)}}{h_d + y \text{ (Experimental)}}$
17	Rye Patch	0.71
8	Alcova	.71
5.2	Boysen (Final)	.72
5.5	Green Mountain	.75
3.1	Scofield	.76
3.6	Bartlett	.76
4.8	Anderson Ranch	.76
5.0	Cachuma	.78
3.4	Falcon	.79
2.8	Unity	.79
2.8	Dickinson	.80
4.5	Medicine Creek (Low)	.80
2.4	Boca	.83
1.3	Fresno	.84
3.7	Boysen (Prelim)	.85
2.5	Caballo	.86

CHUTE SPILLWAY DISCHARGE COEFFICIENTS

MILITARY HYDROLOGY R& D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date ______
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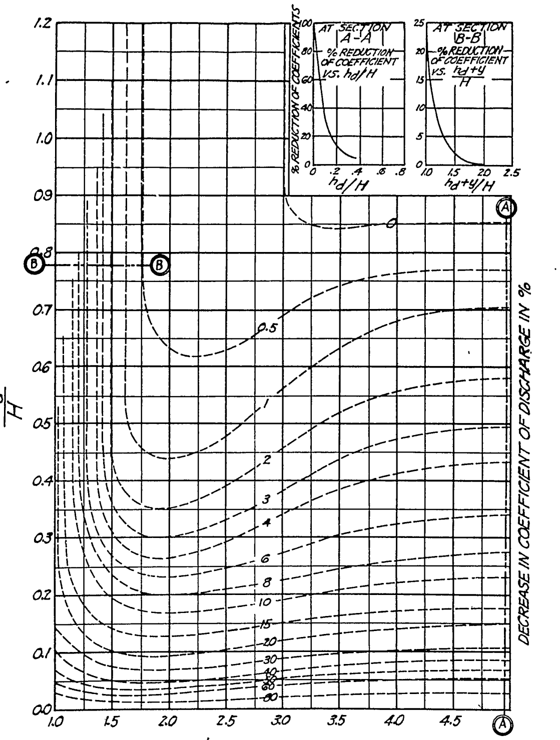

SUBMERGED CREST COEFFICIENTS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
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Drawn by ______

DETERMINATION OF THE DISCHARGE RATING CURVE FOR KACHESS DAM

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(3) The basic data was taken from Plate 615.

DIMENSIONLESS CREST PROFILE

(4) The dimensionless profile was computed by dividing all dimensions by the design head of 8.0 ft. These ratios are tabulated in Col. 2, Table I.

(5) The dimensionless ratios of Col. 2, Table I, were plotted on transparent paper to a scale of 60 units per inch with the design head of 8.0 feet equal to 100 units.

(6) The dimensionless profile was superimposed on Plates 616 and 617. Matching the chute floor immediately downstream from the overfall rather than the co-ordinate axis, the profile of the Kachess Dam spillway agree best with the Fresno, Tiber, and Medicine Creek profiles for an average discharge coefficient of about 3.50.

DISCHARGE RATING CURVE

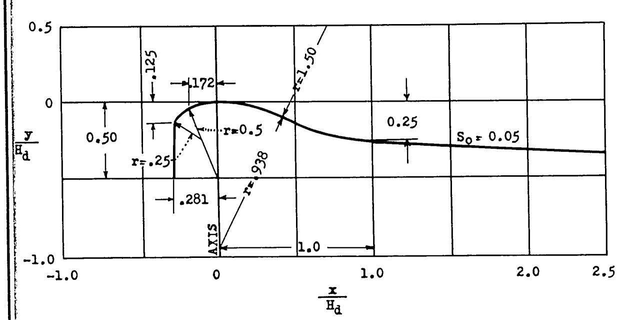
(7) The discharge rating curve was computed as described in the following steps:

1. The head ratios were assumed as shown in Col. 1, Table II.

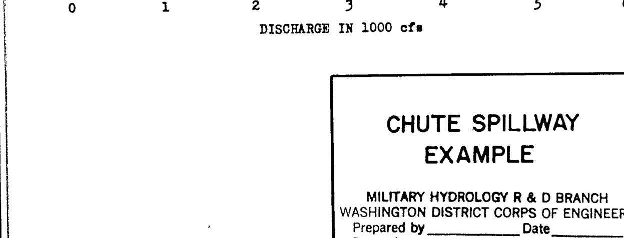
2. The total heads of Col. 2 were determined as the product of the design head (810 ft.) and the ratios of Col. 1, Table II.

3. The coefficient ratios of Col. 3 were determined from the values of Col. 1 and Plate 618. The coefficients of discharge of Col. 4 were computed as the product of the design head coefficient (3.50) and the ratios of Col. 3.

4. The discharge was computed from the data of Cols. 2 and 4 for a spillway width of 50 ft. and entered in Col. 5.



PROFILE OF GATE SECTION



CHUTE SPILLWAY EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date __________
Drawn by ____________

DETERMINATION OF THE DISCHARGE RATING CURVE FOR KACHESS DAM

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-
(3) The basic data was taken from Plate 615.

DIMENSIONLESS CREST PROFILE

(4) The dimensionless profile was computed by dividing all dimensions by the design head of 8.0 ft. These ratios are tabulated in Col. 2, Table I.

(5) The dimensionless ratios of Col. 2, Table I, were plotted on transparent paper to a scale of 60 units per inch with the design head of 8.0 feet equal to 100 units.

(6) The dimensionless profile was superimposed on Plates 616 and 617. Matching the chute floor immediately downstream from the overfall rather than the co-ordinate axis, the profile of the Kachess Dam spillway agree best with the Fresno, Tiber, and Medicine Creek profiles for an average discharge coefficient of about 3.50.

DISCHARGE RATING CURVE

(7) The discharge rating curve was computed as described in the following steps:

1. The head ratios were assumed as shown in Col. 1, Table II.

2. The total heads of Col. 2 were determined as the product of the design head (810 ft.) and the ratios of Col. 1, Table II.

3. The coefficient ratios of Col. 3 were determined from the values of Col. 1 and Plate 618. The coefficients of discharge of Col. 4 were computed as the product of the design head coefficient (3.50) and the ratios of Col. 3.

4. The discharge was computed from the data of Cols. 2 and 4 for a spillway width of 50 ft. and entered in Col. 5.

5. The approach velocity head was computed from the dimensions of the approach channel and the discharge of Col. 5. The flow area of the approach channel was computed as the area from the channel bottom, which was 4.0 feet below the weir crest, to a water surface elevation corresponding to the total head on the weir. The actual water surface elevation would be less than the assumed water surface by an amount equal to the velocity head. The flow area was adjusted and the velocity head recomputed and found to check within 0.1 foot.

6. The difference in the total head of Col. 2 and the velocity head of Col. 6 was added to the elevation of the weir crest (2254.0 ft. msl) to give the reservoir water surface elevation of Col. 7.

(8) The discharge rating curve was plotted from values given in Cols. 5 and 7 of Table II.

DETERMINATION OF THE DISCHARGE RATING CURVE FOR A KACHESS DAM

Item

INITIAL DATA

(1) Profile of the spillway crest of the Kachess Dam. Crest elevation = 2254.0 - Approach channel: Trapezoidal in shape with b = 50 feet at elevation 2250.0 feet msl., and 1 on 2 side slopes.

(2) The design head = 8.0 feet

(3) One spillway gate with 50 foot clear opening.

(4) TABLE I
DIMENSIONLESS CREST PROFILE

Length or Distance Col. 1	x/H_d Col. 2
Upstream face from crest	0.281
Upstream P.T. from crest	0.172
Crest rise from face	0.125
Upstream segment of upstream crest radius	0.25
Downstream segment of downstream crest radius	0.50
Downstream crest radius	0.938
Downstream bucket radius	1.50
Lower P.T. of bucket	1.025

(5) The dimensionless crest profile of the Kachess Dam is shown at the right.

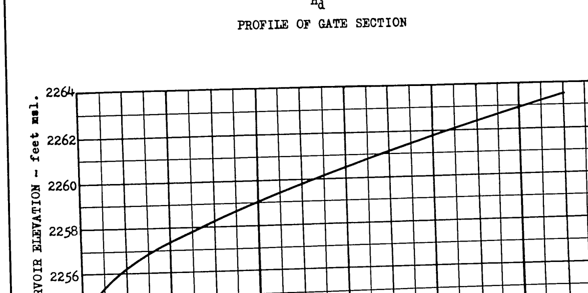
(6) Coefficient of discharge at the design head = 3.50

(7) TABLE II
DISCHARGE RATING CURVE COMPUTATIONS

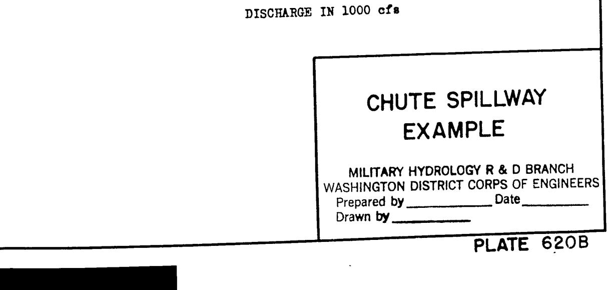
H/H_d (a) Col. 1	H feet Col. 2	C_q/C_d (b) Col. 3	C_q Col. 4	$Q = C_qLH^{1.5}$ cfs Col. 5	$V^2/2g$ feet/sec. Col. 6	Reservoir w.s.elev.* feet msl. Col. 7
0.0	0.00		0.00	0	0.00	2254.0
0.2	1.60	0.910	3.18	321	0.01	2255.6
0.4	3.20	0.948	3.32	948	0.06	2257.1
0.6	4.80	0.974	3.41	1790	0.14	2258.7
0.8	6.40	0.992	3.47	2810	0.22	2260.2
1.0	8.00	1.000	3.50	3950	0.31	2261.7
1.2	9.60	1.003	3.51	5210	0.38	2263.2

(*) Spillway crest elevation = 2254.0 feet msl.
(a) H_d = 8.0 feet.
(b) C_d = 3.50

(8) The discharge rating curve was plotted from values given in Columns 5 and 7 of Table II.



PROFILE OF GATE SECTION



CHUTE SPILLWAY
EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
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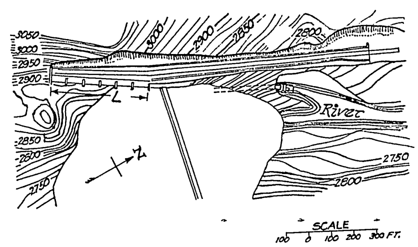
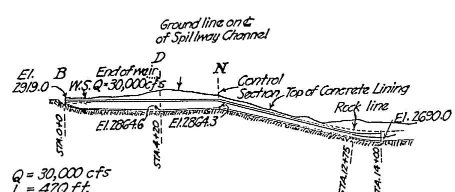


PLAN



$Q = 30,000$ cfs
$L = 420$ ft.
$N = 0.75$
$a = 0.25$

PROFILE

TYPICAL SIDE CHANNEL SPILLWAY

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ________
Drawn by ____________

DETERMINATION OF THE WATER SURFACE PROFILE FOR A SIDE CHANNEL SPILLWAY

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

The assumed physical data for the side channel spillway is shown on Plate 622 E. The total discharge over the spillway crest was assumed to be 4,000 cfs.

DETERMINATION OF CRITICAL DEPTHS AND FRICTION LOSS

(1) The critical depths and friction losses were determined along the spillway channel as described in the following steps:

1. The channel was divided into reaches as shown in Col. 1, Table I, Plate 622 C.

2. The accumulated discharges were entered in Col. 2 for each station. The discharges were based on a unit discharge per foot of width of 20 cfs.

3. The bottom widths of the rectangular channel were determined from the plan shown on Plate 622 E, and entered in Col. 3.

4. The critical depths at the stations of Col. 1 for the discharges of Col. 2 were computed from the equation:

$$Q = 5.67\ b(y_c)^{1.5}$$

as taken from Plate 506. The values of the critical depth were entered in Col. 6.

5. The area and critical velocities were determined from the discharges of Col. 2 and the critical depths of Col. 6, and entered in Cols. 7 and 8 respectively.

6. The hydraulic radius was determined from the areas, bottom widths, and critical depths and entered in Col. 9.

7. The friction losses listed in Col. 10 were determined from the equation

$$h_f = SL$$

Plate 503 was used to compute the slope (S) using the average reach values of the critical velocity and hydraulic radius of Cols. 8 and 9 respectively. The length of each reach was determined from Col. 1.

DETERMINATION OF THE THEORETICAL WATER SURFACE PROFILE AND BOTTOM PROFILE FOR CRITICAL FLOW

(2) The determination of a theoretical water surface profile and bottom profile that would convey the discharge at critical depth was determined as described in the following steps:

1. The values of Cols. 1, 4, 6, 7 and 14 were taken from Table I. The reach lengths were determined from Col. 1 and entered in Col. 2. The increment of inflow in each reach was computed as the product of 20 cfs per foot of spillway length and the length of reach and tabulated in Col. 3.

2. The values of $(Q_1 + Q_2)$ were determined from Col. 4 as the sum of the discharges at the beginning and end of each reach and entered in Col. 5.

3. The values of $(V_1 + V_2)$ were determined from Col. 7 as the sum of the critical velocities at the beginning and end of each reach and entered in Col. 8.

4. The change in velocity in each reach was computed as the difference in the critical velocities of Col. 7 at the beginning and end of each reach and entered in Col. 9.

5. Equation 6-9 was solved in Cols. 10 through 13 from values given in preceding columns, and y entered in Col. 13.

6. The energy loss in each reach was computed as the sum of y and h_f given in Cols. 13 and 14, and entered in Col. 15. The total energy loss at each downstream reach was determined as the sum of the energy loss in the reaches above plus the loss in the reach considered and entered in Col. 16.

7. The theoretical water surface profile was plotted a distance below an arbitrary datum equal to the values given in Col. 16 for each station given in Col. 1. Curve A, Plate 622 E depicts the theoretical water surface profile with the spillway crest used as the datum.

8. The total energy for critical flow at each downstream reach was determined as the sum of the depths of flow listed in Col. 6 and the energy losses given in Col. 16, and entered in Col. 17.

9. The theoretical bottom profile for critical flow was plotted below the arbitrary datum by the values given in Col. 17. Curve B, Plate 622E depicts the bottom profile with the same datum used in step 7.

10. The control section was determined as the point where the actual bottom profile was parallel to the theoretical bottom profile. The control section for the given discharge was station 1 + 50 as shown on Plate 622E.

COMPUTATION OF THE WATER SURFACE PROFILES UPSTREAM AND DOWNSTREAM FROM CONTROL SECTION

(3)-(4) The water surface profiles upstream and downstream from the control point at station 1 + 50 were determined as described in the following steps:

1. The values of the hydraulic properties at the control section (station 1 + 50) were taken from Table I, Plate 622C, and the profile on Plate 622E.

2. A trial change in depth was assumed in Col. 4 and the water surface elevation determined as the sum or difference from the water surface elevations of the downstream or upstream end of the reach respectively and entered in Col. 5.

3. The depth, area, and velocity were determined from the discharge and the cross section and entered in Cols. 6, 7, and 9 respectively.

4. Equation 6-9 was solved in Cols. 10 through 15 inclusive.

5. The friction loss was determined by use of the nomograph on Plate 503 and the reach length, and entered in Col. 17.

6. The total energy loss was determined as the sum of the change in depth of Col. 15 and the friction loss of Col. 17 and entered in Col. 18. The values of Col. 18 were compared with the values of Col. 4. A new trial was made if the values did not check within a reasonable degree of accuracy.

7. The values of the water surface elevations of Col. 5 were plotted for the stations of Col. 1 and shown as curve C on Plate 622E.

Item (1)

TABLE I

DETERMINATION OF CRITICAL DEPTHS

FROM EQUATION $Q = 5.67A(A/b_w)^{0.5}$

Weir discharge (q) is 20 cfs per linear foot of weir

Station Col. 1	Discharge Q cfs Col. 2	Bottom Width b ft. Col. 3	5.67b Col. 4	$\frac{Q}{5.67b}$ Col. 5	Critical depth $y_c = \left(\frac{Q}{5.67b}\right)^{2/3}$ Col. 6	Area $A = by_c$ ft.^{2} Col. 7	Critical Velocity $V_c = \frac{Q}{A}$ Col. 8	Hydr. Radius $R = \frac{A}{P}$ Col. 9	Friction Loss h_f (n=.017) Col. 10
0+00									.03
0+10	200	10.50	59.53	3.36	2.25	23.6	8.47	1.6	.08
0+25	500	11.25	63.79	7.84	3.96	44.5	11.24	2.3	.13
0+50	1000	12.50	70.88	14.11	5.84	73.0	13.70	3.0	.14
0+75	1500	13.75	77.96	19.24	7.20	99.0	15.15	3.5	.14
1+00	2000	15.00	85.05	23.52	8.22	123.3	16.22	3.9	.13
1+25	2500	16.25	92.14	27.13	9.05	147.0	17.00	4.3	.13
1+50	3000	17.50	99.22	30.24	9.70	169.8	17.67	4.6	.13
1+75	3500	18.75	106.31	32.93	10.27	192.5	18.18	4.9	.13
2+00	4000	20.00	113.40	35.27	10.75	~~215.0~~	~~18.60~~	5.2	

(2)

TABLE II

COMPUTATION FOR DETERMINATION OF WATER SURFACE AND BOTTOM PROFILE FOR CRITICAL FLOW

FROM EQUATION 6401: $\Delta y = \frac{Q_1 (V_1 + V_2)}{g (Q_1 + Q_2)} \left(\Delta V + \frac{q\Delta L V_2}{Q_1}\right)$

Station (1)	ΔL ft. (2)	$q\Delta L$ (3)	Q cfs (4)	$(Q_1 + Q_2)$ (5)	y_c ft. (6)	V_c ft./sec (7)	$(V_1 + V_2)$ (8)	ΔV ft./sec. (9)	$\frac{q\Delta LV_2}{Q_1}$ (10)	$\Delta V + \frac{\Delta LqV_2}{Q_1}$ = (9) + (10) (11)	$\frac{Q_1}{g(Q_1+Q_2)}$ (12)	Δy = (8)x(11)x(12) (13)	h_f ft. n = 0.017 (14)	$\Delta y + h_f$ (15)	$\Sigma(\Delta y + h_f)$ (16)	$y_c + \Sigma(\Delta y + h_f)$ = (6) + (16) ft. (17)
0+00			0		0	0							.03	0	0	0
	10	200		200			8.47	8.47						.03	.03	2.28
0+10			200		2.25	8.47							.08			
	15	300		700			19.71	2.77	16.86	19.63	.00887	3.43		3.51	3.54	7.50
0+25			500		3.96	11.24							.13			
	25	500		1500			24.94	2.46	13.70	16.16	.01035	4.17		4.30	7.84	13.68
0+50			1000		5.84	13.70							.14			
	25	500		2500			28.85	1.45	7.58	9.03	.01242	3.24		3.38	11.22	18.42
0+75			1500		7.20	15.15							.14			
	25	500		3500			31.37	1.07	5.41	6.48	.01331	2.70		2.84	14.06	22.28
1+00			2000		8.22	16.22							.13			
	25	500		4500			33.22	.78	4.25	5.03	.01380	2.30		2.43	16.49	25.54
1+25			2500		9.05	17.00							.13			
	25	500		5500			34.67	.67	3.53	4.20	.01411	2.05		2.18	18.67	28.37
1+50			3000		9.70	17.67							.13			
	25	500		6500			35.85	.51	3.03	3.54	.01433	1.82		1.95	20.62	30.89
1+75			3500		10.27	18.18							.13			
	25	500		7500			36.78	.42	2.66	3.08	.01449	1.64		1.77	22.39	33.14
2+00			4000		10.75	18.60										

NOTE: Q_1 and V_1 represent the discharge and velocity at the upstream end of the reach ΔL and Q_2 and V_2 are the same functions for the downstream end. The discharge per linear foot of weir is represented by q.

Item
(3)

TABLE III

COMPUTATION FOR WATER SURFACE CURVE UPSTREAM FROM CONTROL SECTION

$$\text{FROM EQUATION 6401: } \Delta y = \frac{Q_1}{g}\frac{(V_1 + V_2)}{(Q_1 + Q_2)}\left(\Delta V + \frac{q\,\Delta L\,V_2}{Q_1}\right)$$

Station (1)	b ft. (2)	Bottom Elev. ft.msl. (3)	Trial Energy Loss (4)	Water Surf.El. ft.msl. (5)	Depth y ft. (6)	Area A ft.^{2} (7)	Discharge Q cfs (8)	Velocity V ft./sec. (9)	ΔV ft./sec. (10)	$\frac{q \Delta L V_2}{Q_1}$ (11)	$\Delta V + \frac{q \Delta L V_2}{Q_1}$ (12)	$(V_1 + V_2)$ (13)	$\frac{Q_1}{g(Q_1 + Q_2)}$ (14)	Δy = (12) x (13) x (14) (15)	R ft. (16)	h_f (n = 0.017) (17)	$\Delta y + h_f$ ft. (18)	Error (19)
1+50	17.50	78.27		87.97	9.70	170	3000	17.65							4.60			
1+25	16.25	80.27	3.0	90.97	10.7	174	2500	14.37	3.28	3.53	6.81	32.02	.01411	3.08	4.62	.11	3.19	.19
			3.3	91.27	11.0	178.8		14.00	3.65		7.18	31.65		3.21		.11	3.32	.02 OK
1+00	15.00	82.27	2.0	93.27	11.0	165	2000	12.1	1.9	3.5	5.4	26.1	.01381	1.95	4.46	.08	2.03	.03
			2.10	93.37	11.1	166.5		12.0	2.0		5.5	26.0		1.98			2.06	.04
			2.05	93.32	11.05	166		12.05	1.95		5.45	26.05		1.96	4.48	.08	2.04	.01 OK
0+75	13.75	84.75	1.75	95.07	10.32	142	1500	10.55	1.50	4.02	5.52	22.60	.0133	1.66	4.13	.07	1.73	.02 OK
0+50	12.5	87.25	1.48	96.55	9.3	116	1000	8.6	1.95	5.28	7.23	19.15	.01242	1.72	3.74	.05	1.77	.29
			1.69	96.76	9.51	119		8.42	2.13		7.41	18.97		1.74			1.79	.10
			1.80	96.87	9.62	120		8.34	2.21		7.49	18.89		1.75	3.75	.05	1.80	0 OK
0+25	11.25	90.9	1.83	98.7	7.8	87.8	500	5.7	2.64	8.34	10.98	14.04	.01033	1.59	3.27	.02	1.61	.22
			1.61	98.48	7.58	85.3		5.86	2.48		10.82	14.20		1.59	3.23	.02	1.61	0 OK
0+10	10.5	94.25	1.52	100.0	5.75	60.4	200	3.31	2.55	8.8	11.35	9.17	.00887	.92	2.74	.01	.93	.59
			.92	99.40	5.15	54.1		3.7	2.16		10.96	9.56		.93	2.6	.01	.94	.02 OK

(4)

TABLE IV

COMPUTATION FOR DETERMINATION OF WATER SURFACE CURVE DOWNSTREAM FROM CONTROL SECTION

$$\text{FROM EQUATION 6402: } \Delta y = \frac{Q_2}{g}\frac{(V_1 + V_2)}{(Q_1 + Q_2)}\left(\Delta V + \frac{q\,\Delta L\,V_1}{Q_2}\right)$$

Station (1)	b ft. (2)	Bottom Elev. ft.msl. (3)	Trial Energy Loss (4)	Water Surface Elev. ft.msl. (5)	Depth y ft. (6)	Area A ft.^{2} (7)	Discharge Q cfs (8)	Velocity V ft./sec. (9)	ΔV ft./sec. (10)	$\frac{q \Delta L V_1}{Q_2}$ (11)	$(\Delta V + \frac{q \Delta L V_1}{Q_2})$ (12)	$(V_1 + V_2)$ (13)	$\frac{Q_2}{g(Q_1+Q_2)}$ (14)	Δy = (12) x (13) x (14) (15)	R ft. (16)	h_f (17)	$\Delta y + h_f$ ft. (18)	Error (19)
1+50	17.50	78.27		87.97	9.70	170	3000	17.65							4.6			
1+75	18.75	74.27	5.50	82.47	8.2	154	3500	22.75	5.10	2.52	7.62	40.40	.0167	5.15	4.4	.18	5.33	.17
			5.6	82.37	8.1	152		23.0	5.35		7.87	40.65		5.35		.18	5.53	.07
			5.65	82.32	8.05	151		23.18	5.53		8.05	40.83		5.51		.18	5.69	.04
			5.63	82.34	8.07	151.31		23.13	5.48		8.00	40.78		5.45	4.3	.19	5.64	.01 OK
2+00	20.00	70.27	4.07	78.27	8.0	160	4000	25.0	1.87	2.89	4.76	48.13	.01655	3.80	4.45	.25	4.05	.02 OK

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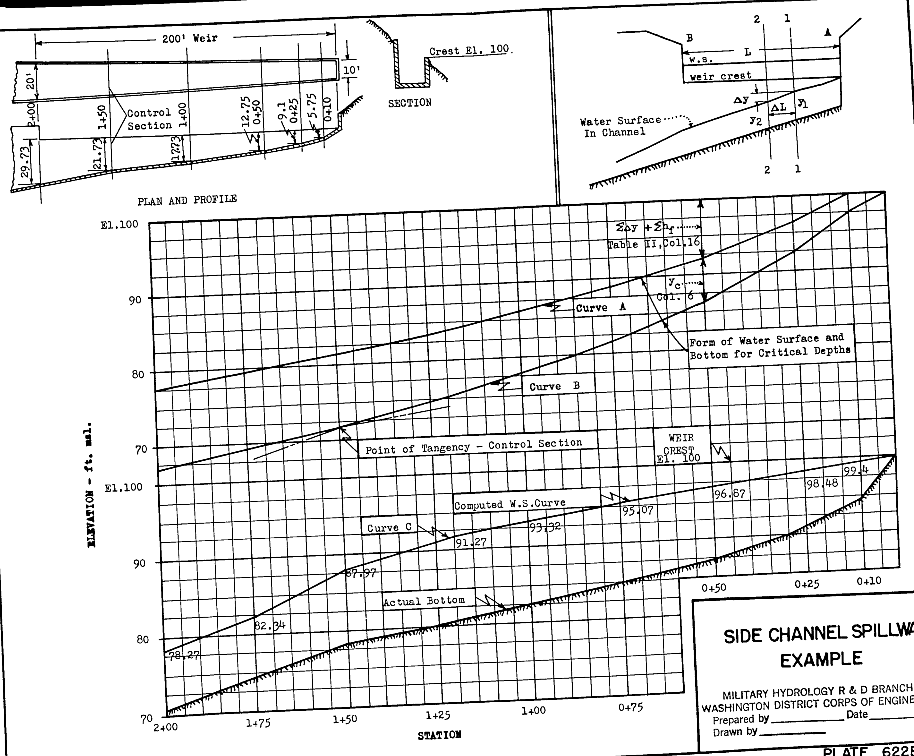
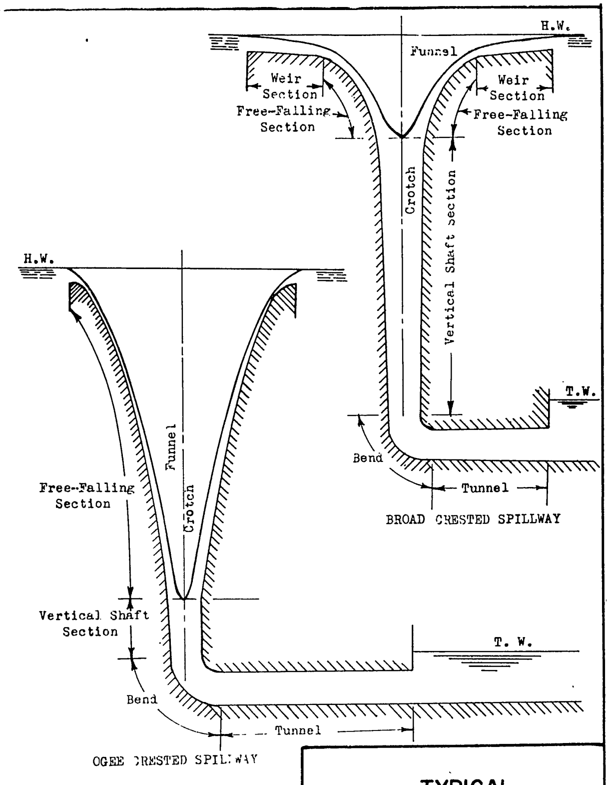


SIDE CHANNEL SPILLWAY EXAMPLE

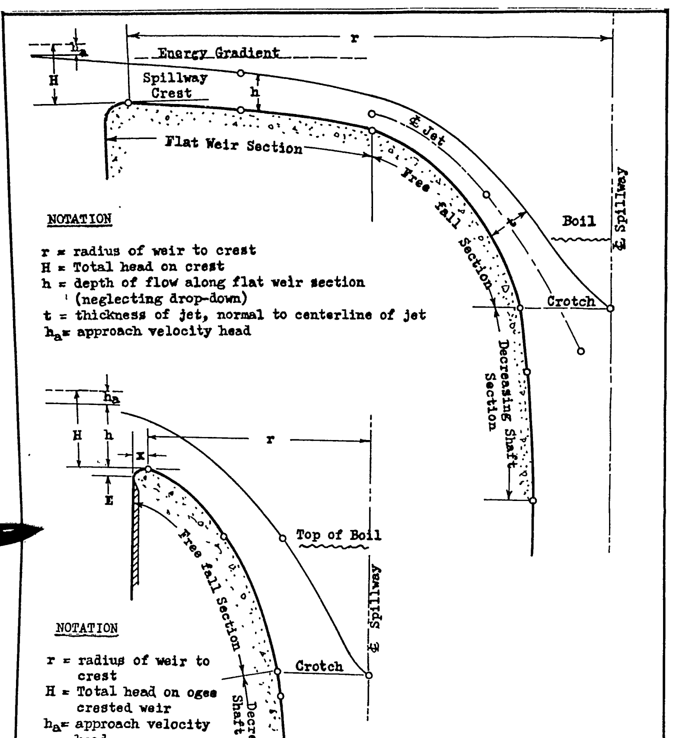
MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by __________ Date __________
Drawn by __________

PLATE 622E



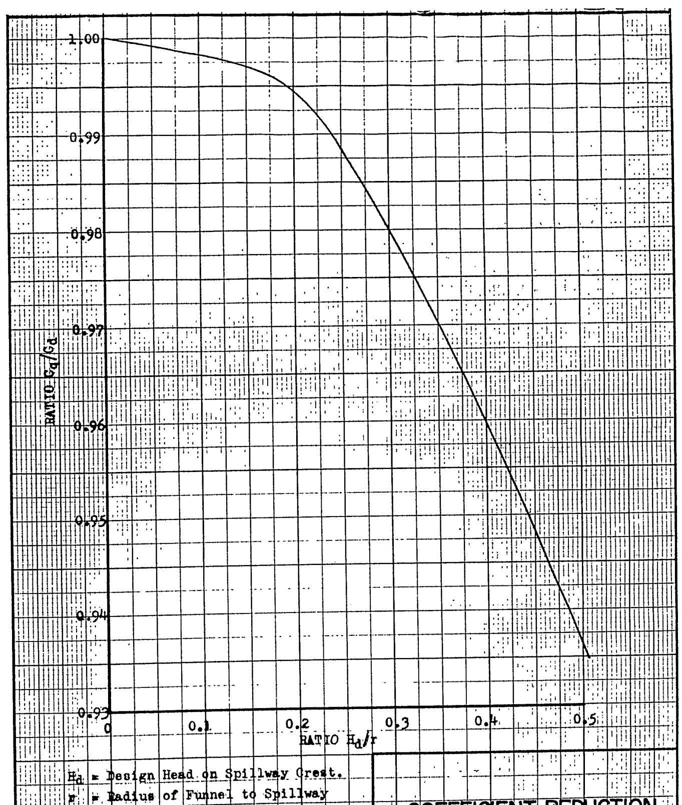
TYPICAL MORNING GLORY SPILLWAY

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by __________ Date __________
Drawn by __________



MORNING GLORY SPILLWAY CRESTS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______________ Date __________
Drawn by ______________



COEFFICIENT REDUCTION FOR RADIAL FLOW

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______________ Date __________
Drawn by ______________

DETERMINATION OF THE DISCHARGE RATING CURVE FOR AN OGEE CREST MORNING GLORY SPILLWAY

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(8) Assumed physical data.

(9) The effects of the convergence of the flow filaments for the morning glory spillway were determined from Plate 625. The ratio of the design head and the radius of curvature was determined from items (8) and (1) of the initial data. The ratio of the design discharge coefficient of the morning glory spillway to the design discharge coefficient of an ogee spillway was found to be 0.977. The coefficient of discharge of the ogee crest morning glory spillway at the design head was computed as 0.977 x 3.97 = 3.88.

WEIR DISCHARGE RATING CURVE

(10) Table I was computed in the same manner as described in item (7) on Plate 614 with $C_d = 3.88$ and design head of 15 ft.

DISCHARGE RATING CURVE FOR OUTLET CONDUIT

(11) The upper segment of the discharge rating curve of the ogee-crest morning glory spillway was computed as described in the following steps:

1. Several discharges were assumed as given in Col. 1, Table II. Discharges were selected that appeared to require a greater total head and a less total head under pipe flow conditions than under weir conditions for the same discharge.

2. The outlet velocity and velocity head were determined from the discharges of Col. 1 and the area of the outlet conduit of item (2) and entered in Cols. 2 and 3 respectively.

3. The total head on the spillway was determined in terms of the outlet velocity head as follows:

$$H = h_f + h_b + V_o^2/2g$$

<u>Friction Loss</u>

$$h_f = K_f\ V_o^2/2g$$

$$K_f = C_f\ L/R^{4/3}$$

$$C_f = 2gn^2/2.208 = 0.00492$$

$$R = A/P = D/4 = 7.14 \quad R^{4/3} = 7.14^{4/3} = 13.8$$

$$L = 660 \text{ ft.}$$

$$K_f = (0.00492)\ (660)/13.8 = 0.23$$

<u>Bend Loss</u>

$$h_b = K_b\ V_o^2/2g$$

$$K_b = \frac{1}{(\log_e r/D + \quad /2)}$$

$$K_b = 0.14$$

<u>Total Head</u>

Exit	= 1.000
Friction	= 0.236
Bend	= 0.140
Total	= 1.376 $V_o^2/2g$

The total head was computed as the product of the velocity heads in Col. 3 and 1.376 and entered in Col. 4.

4. The head on the weir was computed as the difference between the heads of Col. 4 and 135.8 and entered in Col. 5.

5. The heads of Col. 5 were added to the weir crest elevation to give the reservoir water surface elevation and entered in Col. 6.

(12) The water surface elevations and discharge data of Tables I and II were plotted. Below the intersection of the curves the weir controlled the discharge, above the point of intersection the conduit controlled.

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DETERMINATION OF THE DISCHARGE RATING CURVE FOR AN OGEE CREST MORNING GLORY SPILLWAY

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(8) Assumed physical data.

(9) The effects of the convergence of the flow filaments for the morning glory spillway were determined from Plate 625. The ratio of the design head and the radius of curvature was determined from items (8) and (1) of the initial data. The ratio of the design discharge coefficient of the morning glory spillway to the design discharge coefficient of an ogee spillway was found to be 0.977. The coefficient of discharge of the ogee crest morning glory spillway at the design head was computed as 0.977 x 3.97 = 3.88.

WEIR DISCHARGE RATING CURVE

(10) Table I was computed in the same manner as described in item (7) on Plate 614 with C_d = 3.88 and design head of 15 ft.

DISCHARGE RATING CURVE FOR OUTLET CONDUIT

(11) The upper segment of the discharge rating curve of the ogee-crest morning glory spillway was computed as described in the following steps:

1. Several discharges were assumed as given in Col. 1, Table II. Discharges were selected that appeared to require a greater total head and a less total head under pipe flow conditions than under weir conditions for the same discharge.

2. The outlet velocity and velocity head were determined from the discharges of Col. 1 and the area of the outlet conduit of item (2) and entered in Cols. 2 and 3 respectively.

3. The total head on the spillway was determined in terms of the outlet velocity head as follows:

$$H = h_f + h_b + V_o^2/2g$$

Friction Loss

$$h_f = K_f V_o^2/2g$$

$$K_f = C_f L/R^{4/3}$$

$$C_f = 2gn^2/2.208 = 0.00492$$

$$R = A/P = D/4 = 7.14 \quad R^{4/3} = 7.14^{4/3} = 13.8$$

$$L = 660 \text{ ft.}$$

$$K_f = (0.00492)(660)/13.8 = 0.23$$

Bend Loss

$$h_b = K_b V_o^2/2g$$

$$K_b = \frac{1}{(\log_e r/D + \quad /2)}$$

$$K_b = 0.14$$

Total Head

Exit	= 1.000
Friction	= 0.236
Bend	= 0.140
Total	= 1.376 $V_o^2/2g$

The total head was computed as the product of the velocity heads in Col. 3 and 1.376 and entered in Col. 4.

4. The head on the weir was computed as the difference between the heads of Col. 4 and 135.8 and entered in Col. 5.

5. The heads of Col. 5 were added to the weir crest elevation to give the reservoir water surface elevation and entered in Col. 6.

(12) The water surface elevations and discharge data of Tables I and II were plotted. Below the intersection of the curves the weir controlled the discharge, above the point of intersection the conduit controlled.

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DETERMINATION OF THE DISCHARGE RATING CURVE FOR OGEE CREST MORNING GLORY SPILLWAYS

COMPUTATIONS

INITIAL DATA

Item		
(1)	Funnel diameter (2r)	= 90.0 ft.
(2)	Diameter of vertical shaft and outlet conduit	= 28.5 ft.
(3)	Radius of 90° bend in outlet conduit	= 60.0 ft.
(4)	Length of spillway shaft	= 660.0 ft.
(5)	Elevation of crest	= 3255.0 ft. msl.
(6)	Elevation of outlet invert	= 3105.0 " "
(7)	Coefficient of friction (Manning's "n")	= 0.013
(8)	H_d (Determined from Plate 6101)	= 15 ft.
(9)	Coefficient of discharge at the design head	= 3.88

(10) TABLE I
WEIR DISCHARGE RATING CURVE COMPUTATIONS

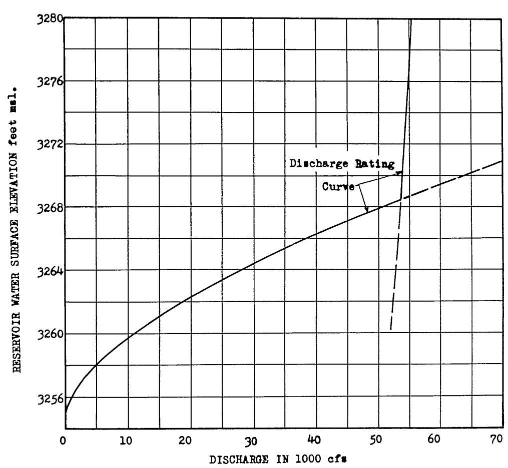
H/H_d Col. 1	H feet Col. 2	Reservoir W.S. El. ft. msl. Col. 3	C_q/C_d Col. 4	C_q Col. 5	$Q = C_q L H^{1.5}$ cfs Col. 6
0.0	0	3255.0	–	–	0
0.2	3.0	3258.0	0.85	3.30	4,850
0.4	6.0	3261.0	0.90	3.49	14,500
0.6	9.0	3264.0	0.94	3.64	27,800
0.8	12.0	3267.0	0.975	3.78	44,400
1.0	15.0	3270.0	1.00	3.88	63,700
1.2	18.0	3273.0	1.025	3.97	85,800

(11) TABLE II
CONDUIT DISCHARGE RATING COMPUTATIONS

Q cfs Col. 1	$V_o = Q/A$ ft/sec Col. 2	$\frac{V_o^2}{2g}$ feet Col. 3	$H = 1.376\frac{V_o^2}{2g}$ ft. Col. 4	Head on Weir ft. (a) Col. 5	Reservoir W.S. El. ft. msl. Col. 6
52,000	81.5	103	141	5.8	3260.8
55,000	86.1	115	158	22.2	3277.2

(a) Difference in elevation of weir crest and center line of outlet conduit = 135.8 feet.

Head on weir = H — 135.8



MORNING GLORY SPILLWAY
OGEE WEIR

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ________
Drawn by ____________

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EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(6) Assumed physical data.

DISCHARGE RATING CURVE FOR BROAD-CRESTED WEIR

(7) The heads given in Col.1, Table I were assumed and raised to the 1.5 power and entered in Col.2. The discharge over the weir was computed as follows:

$$Q = C_q L H^{1.5}$$

where

$C_q = 3.00$ (See Paragraphs 126 & 127).

$L = \pi D = 180.0$

$D = 2r$ as shown on Plate 624.

$Q = 3.00 \times 180.0\pi \times H^{1.5} = 1700H^{1.5}$

The discharge was computed as the product of 1700 and the values of Col.2 and entered in Col.3.

DISCHARGE RATING CURVE FOR OUTLET CONDUIT

(8) The upper segment of the discharge rating curve of the broad-crested morning glory spillway was computed in the same manner as given in item (11), Plate 626.

(9) The head-discharge data of Tables I & II were plotted in the same manner as given in Item (12), Plate 626.

DETERMINATION OF THE DISCHARGE RATING CURVE FOR BROAD CRESTED MORNING GLORY SPILLWAYS

COMPUTATIONS

Item

INITIAL DATA

Item		
(1)	Funnel diameter (2r)	= 180.0 feet
(2)	Diameter of vertical shaft and outlet conduit	= 22.5 "
(3)	Radius of bend	= 55.0 "
(4)	Length of conkuit from crest to outlet	= 323.0 "
(5)	Difference in elevation crest to center line of outlet conduit	= 169.0 "
(6)	Coefficient of friction (Manning's "n")	= 0.013

(7) TABLE I
WEIR DISCHARGE RATING COMPUTATIONS

H (ft.) Col.1	$H^{1.5}$ Col.2	$Q = 1700H^{1.5}$ (cfs) Col.3
1	1.0	1,700
3	5.2	8,340
5	11.2	19,000
7	18.5	31,400
10	31.6	53,700

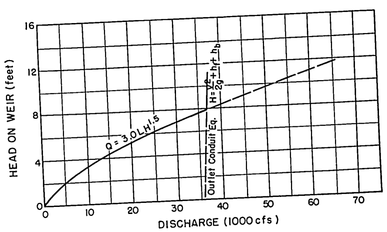
(8) TABLE II
CONDUIT DISCHARGE RATING COMPUTATIONS

Q (cfs) Col.1	$V_o = Q/A$ (ft/sec) Col.2	$\frac{V_o^2}{2g}$ Col.3	$H = 1.33\frac{V_o^2}{2g}$ (feet) Col.4	Head on Weir (feet)(a) Col.5
36,000	90.5	127	169	0.0
37,000	93.3	134	178	9.0

(a) Head on weir = H - 169.0

NOTE:

The reservoir water surface elevation would be determined by adding the head from the discharge rating curve to the weir crest elevation.



MORNING GLORY SPILLWAY
BROAD CREST WEIR

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS

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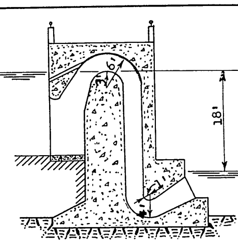
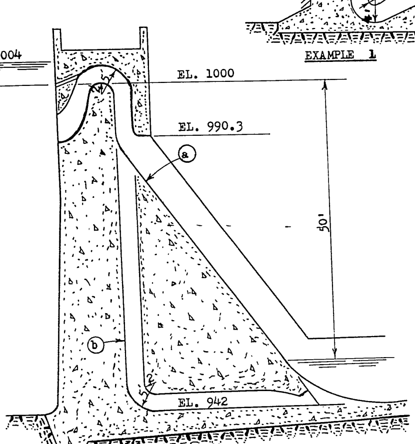


EXAMPLE 1



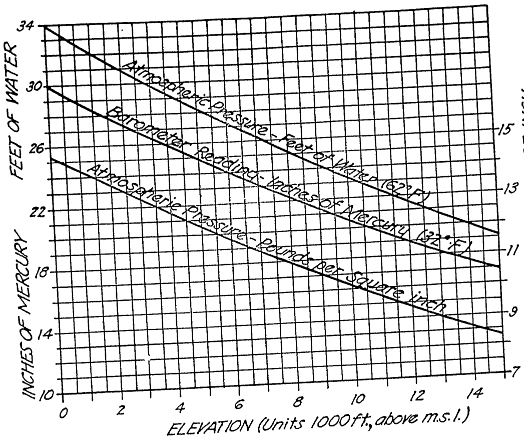
EXAMPLE 2

SIPHON SPILLWAYS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date _________
Drawn by ___________

MHB-12

PLATE 628



NATURAL LOGARITHMS

N	Log_e	N	Log_e	N	Log_e
1.0	0.000	3.0	1.098	6.5	1.872
1.2	0.182	3.2	1.163	7.0	1.946
1.4	0.336	3.4	1.224	7.5	2.015
1.6	0.470	3.6	1.281	8.0	2.078
1.8	0.588	3.8	1.335	8.5	2.140
2.0	0.693	4.0	1.386	9.0	2.197
2.2	0.788	4.5	1.504	9.5	2.251
2.4	0.875	5.0	1.609	10.0	2.303
2.6	0.955	5.5	1.705	11.0	2.398
2.8	1.029	6.0	1.792	12.0	2.485

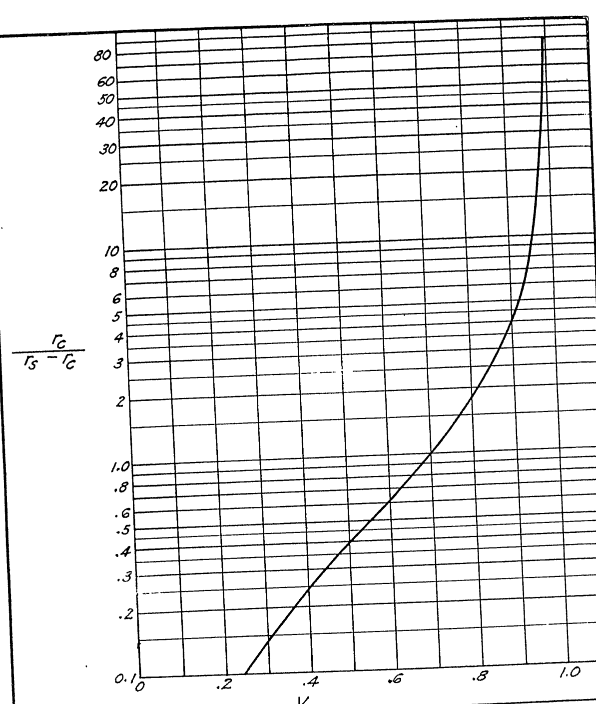
N = Number
Log_e = Natural logarithm

ENGINEERING DATA FOR SIPHONS

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ______ Date __________
Drawn by ________.

PLATE 629

MHB-12



NOTE:

r_c = radius of throat crest
r_s = " " " summit
V = Mean throat velocity
V_{cr} = Crest throat velocity

SIPHON SPILLWAY
THROAT RADIUS VS.
THROAT VELOCITY

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by______ Date ______
Drawn by ______

MHB - 12

PLATE 630

DETERMINATION OF THE DISCHARGE
OF A LOW HEAD SIPHON

EXPLANATION OF COMPUTATIONS

<u>Item</u>

INITIAL DATA

(1)-(9) Assumed physical data for a low head siphon.

HEAD LOSS COEFFICIENTS

(10) <u>Entrance Loss Coefficient</u> (K_e). A conservative value of the entrance loss for the siphon was assumed to be 0.10 and was determined from Plate

(11) <u>Friction Loss Coefficient</u> (K_f). The friction loss coefficient was determined as described in the following steps:

1. The center line length of the siphon was determined as the sum of the arc lengths of the two bends and the length of the vertical leg. The arc lengths were determined as the product of the ratio of the deflection angle to 360°, and the circumference of a circle with a radius equal to the radius of the bend.

   Upper bend $\frac{150}{360}$ 2 4.5 = 13.1 ft.

   Vertical leg = 19.0 ft.

   Lower bend $\frac{120}{360}$ 2 2.5 = <u>5.2 ft.</u>

   Total friction length = 37.3 ft.

2. The friction head coefficient was computed by Eq. 4-9 with C_f determined from the tabulated values in Par. 48 for n = 0.012.

   $R = 15/16 = 0.938 \quad R^{4/3} = 0.919$

   $$K_f = \frac{C_f L}{R^{4/3}} = \frac{(0.00419)\ (37.3)}{0.919}$$

   $K_f = 0.17$

(12)-(13) <u>Bend Loss Coefficient</u> (K_b). The upper and lower bend loss coefficients were computed by Eq. 6-22 as follows:

$$K_b = 0.23\ (r_s - r_c)/r_c$$

Upper $K_b = 0.23\ (6 - 3)/3$

Plate 631 A

$K_b = 0.23$

Lower $K_b = 0.23\ (4 - 1)/1$

$K_b = 0.69$

(14) Gradual Expansion Coefficient (K_{ge}). The gradual expansion coefficient was computed by Eq. 4-20 as follows:

$$K_{ge} = (1 - A_t/A_o)^2 \text{ Sin } \theta$$

$$K_{ge} = 0.01$$

(15) Outlet Velocity Head. The outlet velocity head coefficient was expressed in terms of the throat velocity by Eq. 4-36 as follows:

$$K = (A_t/A_o)^2 K_o$$

$$= (15/20)^2 \times 1.0$$

$$K = 0.56$$

(16) The sum of the velocity head coefficients in terms of the throat velocity was determined from items (10) through (15).

DISCHARGE COMPUTATION

(17) The discharge from the siphon with an 18 foot head was computed by Eq. 6-19 using the friction and form losses given in item (16) and the throat area given in item (4).

DETERMINATION OF THE DISCHARGE

OF A LOW HEAD SIPHON

Item

INITIAL DATA

(1) Siphon illustrated on Plate 628, Example 1.

(2) Normal head 18'

(3) Entrance cross-sectional area = 5' x 8' = 40 feet ^{2}

(4) Throat cross-sectional area = 3' x 5' = 15 feet ^{2}

(5) Leg cross-sectional area = 3' x 5' = 15 feet ^{2}

(6) Outlet cross-sectional area = 4' x 5' = 20 feet ^{2}

(7) Upper bend: r_c = 3 feet, r_s = 6 feet, deflection angle = 150°

(8) Lower bend: r_1 = 1 foot, r_2 = 4 feet, deflection angle = 120°

(9) Manning's roughness coefficient = 0.012.

HEAD LOSS COEFFICIENTS

(10)	Entrance Loss	= 0.10
(11)	Friction Loss	= 0.17
(12)	Upper Bend Loss	= 0.23
(13)	Lower Bend Loss	= 0.69
(14)	Gradual Expansion Loss	= 0.01
(15)	Outlet Velocity Head	= 0.56
(16)	Total Head	= 1.76 $V^2/2g$

DISCHARGE COMPUTATION

(17) $$Q = A_t \left[\frac{2gH}{\Sigma K_s + K_o}\right]^{0.5} = 15 \left[\frac{2g \times 18}{1.76}\right]^{0.5} = 385 \text{ cfs}$$

SIPHON SPILLWAY

EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ________
Drawn by ____________

DETERMINATION OF THE DISCHARGE OF A LOW HEAD SIPHON

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(11) Assumed physical data for a low head siphon.

HEAD LOSS COEFFICIENTS

(12) Entrance Loss Coefficient (K_e). The entrance loss was determined in the same manner as item (10), Plate 631.

(13) Friction Loss Coefficient (K_f). The friction loss coefficient was determined in the same manner as item (11), Plate 631.

1. Friction length:

Upper bend	$= \frac{170^\circ}{360^\circ} 2\pi 3.5$	= 10.4 ft.
Vertical leg		= 9.0 ft.
Total friction length		= 19.4 ft.

2. Friction loss coefficient:

$$K_f = \frac{C_f L}{R^{4/3}} = \frac{(0.00419)\ (19.4)}{0.919}$$

$$K_f = 0.089$$

(14) Upper Bend Loss Coefficient (K_b). The bend loss coefficient was determined in the same manner as item (12), Plate 631.

$$K_b = \frac{0.23\ (5 - 2)}{2}$$

$$K_b = 0.35$$

(15) Outlet Velocity Head. The outlet velocity head coefficient was determined as unity from Par. 57. The cross-sectional area of the siphon throat was equal to the outlet area.

(16) The sum of the velocity head coefficients was determined from items (12) and (15).

DISCHARGE COMPUTATIONS

(17) The discharge from the low head siphon was computed by Eq. 6-19 from a head of 13.7 ft. The losses were given in item (16) and the throat area was given in item (4).

DETERMINATION OF THE DISCHARGE OF A LOW HEAD SIPHON

Item

INITIAL DATA

(1) Siphon illustrated on Plate 628, Example 2a.

(2) Normal head = 13.7 feet

(3) Entrance cross-sectional area = 5' x 8' = 40 square feet

(4) Throat cross-sectional area = 3' x 5' = 15 square feet

(5) Leg cross-sectional area = 3' x 5' = 15 square feet

(6) Throat crest elevation = 1000 feet msl.

(7) Maximum pool elevation = 1004 feet msl.

(8) h_a at elevation 1000 ft. msl. = 32.65 feet

(9) Upper bend r_c = 2 feet, r_s = 5 feet, deflection angle = 170°

(10) Free outlet at elevation 990.3 feet msl.

(11) Manning's roughness coefficient = 0.012

HEAD LOSS COEFFICIENTS

(12)	Entrance Loss	= 0.10
(13)	Friction Loss	= 0.089
(14)	Upper Bend Loss	= 0.35
(15)	Outlet Velocity Head	= 1.00
(16)	Total Head	= 1.54 $V^2/2g$

DISCHARGE COMPUTATION

(17) $$Q = A_t \left[\frac{2gH}{\Sigma K_s + K_o}\right]^{0.5} = 15\left[\frac{2g \times 13.7}{1.54}\right]^{0.5} = 360 \text{ cfs}$$

SIPHON SPILLWAY EXAMPLE

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DETERMINATION OF THE DISCHARGE
OF A HIGH HEAD SIPHON

EXPLANATION OF COMPUTATIONS

Item	
	INITIAL DATA
(1)-(7)	Assumed physical data for a high head siphon.
	HEAD LOSS COEFFICIENTS
(8)	Entrance Loss Coefficient (K_e). The entrance loss coefficient was determined in the same manner as item (10), Plate 631.
(9)	Friction Loss Coefficient (K_f) The friction loss coefficient was determined in the same manner as item (11), Plate 631. 1. Friction length = 94.0 ft. 2. Friction loss coefficient. $K_f = \frac{C_f L}{R^{4/3}} = \frac{(0.00419)\ (94.0)}{0.919}$; $K_f = 0.43$
(10)-(11)	Bend Loss Coefficient (K_b). The upper and lower bend loss coefficients were identical to item (14), Plate 632. $K_b = 0.35$
(12)	Outlet Velocity Head. The outlet velocity head was expressed in terms of the throat velocity by Eq. 4-36 as follows: $K = (A_t/A_o)^2 K_o = (15/6.85)^2 \times 1.0 = 4.80$
(13)	The sum of the velocity head coefficients was determined from items (8) through (12).
	DISCHARGE COMPUTATION
(14)	The discharge from the high head siphon was computed by Eq. 6-19 for a head of 54.0 ft. The structure and outlet losses were given in item (13) and the throat area was given in item (2).

DETERMINATION OF THE DISCHARGE

OF A HIGH HEAD SIPHON

Item	
	INITIAL DATA
(1)	Siphon illustrated on Plate 628, Example 2b.
(2)	Physical data listed in Items (3), (4), (5), (6), (7), (8), (9), and (11) on Plate 632.
(3)	Lower bend identical with upper bend.
(4)	Normal head = 54.0 feet
(5)	Restricted outlet at elevation 942.0 feet msl.
(6)	Restricted cross-sectional area = 6.85 square feet.
(7)	Friction length = 94.0 feet.

HEAD LOSS COEFFICIENTS

Item		
(8)	Entrance Loss	= 0.10
(9)	Friction Loss	= 0.43
(10)	Upper Bend Loss	= 0.15
(11)	Lower Bend Loss	= 0.35
(12)	Outlet Velocity Head	= 4.80
(13)	Total Head	= 6.03 $V^2/2g$

DISCHARGE COMPUTATIONS

(14) $$Q = A_t \left[\frac{2gH}{\Sigma K_s + K_o}\right]^{0.5} = 15 \left[\frac{2g \times 54}{6.03}\right]^{0.5} = 360 \text{ cfs}$$

SIPHON SPILLWAY
EXAMPLE

CHAPTER VII
NAVIGATION DAMS

SECTION A: GENERAL CONSIDERATIONS

137. Classification of Navigation Dams. a. Definition. A navigation dam is a barrier constructed across a river channel to provide an increase in depth for navigation. A navigation dam is normally low in height with its spillway crest or sill at or near the channel bottom. In contrast,a flood control or power dam is normally constructed across the entire river valley and is relatively high with its crest considerably above the valley floor.

b. Crest Types. The crests of dams used in canalization /1/ can be classified as fixed, movable (non-navigable or navigable), and fixed with movable gates.

(1) Fixed Crest. A navigation dam with a fixed crest is normally a weir constructed to the height necessary to provide the navigable depth at the shallowest point upstream. It is an uncontrolled weir and the discharge is computed by methods described in chapters III and VI.

(2) Movable, Non-Navigable Crest. A dam with a movable non-navigable crest consists of a very low concrete sill on which gates are superposed. Each gate may be raised or lowered, but the crest will not permit passage of vessels without the use of a lock.

(3) Movable, Navigable Crest. The movable, navigable dam consists of a very low sill upon which is superposed a wicket or gate structure to control the upstream water surface, but which when collapsed allows passage of vessels upstream without the use of a lock.

(4) Fixed Weir With Movable Crest. The fixed dam with a movable crest is a compromise between a fixed crest dam and a movable crest dam that is non-navigable. The crest is not as high as a fixed crest dam, yet higher than the sill of the movable crest dam. It acts as a gated submerged weir.

138. Crest Control of Navigation Dams. The crest or sills of navigation dams /1/ often are controlled by means of collapsible wickets or gates.

a. Collapsible Controls. The collapsible type of crest control is used on crests of movable, navigable, and non-navigable types of dams, as described above. The controls would normally consist of Chanoine wickets for the navigable pass and Chanoine, or Bebout wickets, Boule trestles, or beartrap gates for the regulating weir. Each of these crest controls collapses on the sill during flood flows, and does not appreciably obstruct the flood passage. The "swellhead" or rise in the upstream water surface under flood conditions would be computed by the methods described in par. 140. Pictures and descriptions of the various types of wickets and gates used as collapsible controls are given on Figures 1 to 3, inclusive, on Plate 701.

b. Gate Controls. The gate type of crest control is used on crests of movable, non-navigable types of dams. The gates are of four general types: roller gates, tainter gates (nonsubmersible and submersible), vertical lift gates and Sidney type gates. Each of these gates are fixed between piers on the crest or sill of the dam and moves vertically to pass regulation or flood flows under the gate. The roller gate and submersible tainter gate are designed to pass moderate regulation flows over, as well as under, the gates. Pictures and descriptions of the various types of gates are given on Figures 4 to 7, inclusive, on Plate 701. Discharges from the gates on a navigation dam would be computed for conditions of full and partial gate opening.

SECTION B: CREST GATES FULLY OPEN

139. Applications. Low head barrages such as diversion and navigation dams usually discharge their outflows at or near riverbed levels through a series of gates separated by masonry piers. The gates are normally fully opened for the passage of flood flows. The head required above tailwater to establish flow through the structure is equal to the hydraulic loss through the structure. The hydraulic losses are caused by contraction of the flow at the abutments and at the piers, friction of the flow on the pier walls, expansion of the flow into the channel downstream, and (in the case of gate spillways) turbulence at the gate and stop log recesses. A discharge rating curve is determined from the tailwater rating curve at the structure and the use of D'Aubuisson's or Nagler's Formula to compute the head loss through the structure.

140. Discharge Capacity. a. The discharge capacity of a fully opened gate is determined by adding the tailwater elevation, and the head loss through the structure for each discharge. The head loss through the structure would be determined by the equation:

$$H_L = h_e + h_f + h_o + h_r + h_g \qquad (7\text{-}1)$$

where

H_L = the total head loss through the structure
h_e = the head loss at the entrance including the loss due to contraction at the upstream ends of the piers
h_f = the loss due to friction on the piers
h_o = the head loss at the exit of the structure
h_r = the head loss due to stop log recesses (taken as 0.05 velocity head through the structure)
h_g = the head loss due to gate recesses (taken as 0.10 velocity head through the structure)

b. The D'Aubuisson or Nagler formula /2/ & /3/ would take care of the entrance, friction, and exit losses. Therefore, the total head loss would be

$$H_L = h_d + h_r + h_g \qquad (7\text{-}2)$$

where

h_d = the head loss computed by the D'Aubuisson or Nagler equation.

c. The D'Aubuisson formula is not applicable to cases where the swellhead is greater than 0.2 times the depth of water above the weir(measured to the sill elevation). For swellheads greater than this, the basic weir formula, $Q = C_q L H^{1.5}$ is applicable. The D'Aubuisson formula should be used only where the flow is subcritical. The Nagler formula may be used where the flow is subcritical, critical and even supercritical.

The D'Aubuisson Equation:

$$Q = K_D\, b_2 y_3\ (2gh_d + V_1^2)^{0.5} \qquad (7\text{-}3)$$

or if $y_2 = y_3$

$$h_d = \left(\frac{1}{K_D^2}\right) V_2^2/2g - V_1^2/2g \qquad (7\text{-}4)$$

The Nagler Equation:

$$Q = K_N\, b_2\, (2g)^{0.5}\ (y_3 - 0.3\, V_3^2/2g)\ \ (h_d + KV_1^2/2g)^{0.5} \qquad (7\text{-}5)$$

where

Q = the quantity of water flowing in cubic feet per second.
y_1 = the mean depth of the water upstream from the nose of the pier at a distance equal to the length of the pier.
y_2 = the mean depth of the stream in the most contracted section of the opening.
y_3 = the mean depth of the water in the channel below the contraction; that is, the depth in the unobstructed channel.
b_1 = the mean width of the channel above the contraction.
b_2 = the mean width of flow at the most contracted section of the opening.
b_3 = the mean width of the channel below the contraction, ordinarily equal to b_1.

V_1 = the mean velocity of the water above the contraction = $Q/b_1\ y_1$
V_2 = the mean velocity of the water in the most contracted section of the channel = $Q/b_2\ y_2$
V_3 = the mean velocity of the water in the channel below the contraction = $Q/b_3\ y_3$
h_d = the drop of the water surface in passing through the contraction = $y_1 - y_3$
g = the acceleration of gravity.

A_r = channel contraction ratio = $\frac{\text{cross-sectional area of obstruction}}{\text{cross-sectional area of channel}}$
K_D = a pier-shape coefficient to take account of the losses due to friction, impact, eddies, etc., in D'Aubuisson's formula, see Plate 703.
K_N = Nagler pier shape coefficient, see Plate 703.
K = velocity head correction coefficient, Plate 702.

d. Factors affecting K_D and K_N /4/, see Plate 705, are:
(1) The shape of the pier nose and tail.
(2) The percentage of channel contraction caused by the pier.
(3) The length of the pier.
(4) The angle which the pier makes with the thread of the stream. For angles of 10° or less, there is no difference in the pier less coefficient. With the pier at a 20° angle to the current, a decrease of about 7 percent in either the D'Aubuisson or Nagler coefficient will result.
(5) Dam abutments, sills and gate-recesses. The net effect of the combined obstruction losses due to pier recesses, locks, non-overflow sections and low sills amounted to 11.4 percent reduction in either the D'Aubuisson or Nagler coefficients for a number of the Upper Mississippi River navigation dams.

e. The total head loss through the structure for subcritical flow is computed by Equation 7-2, with Equation 7-4 used to compute h_d. The head lost through the structure is

$$H_L = \left(\frac{1}{K_D^2}\right) \frac{V_2^2}{2g} + 0.15\ \frac{V_2^2}{2g} - \frac{V_1^2}{2g} \qquad (7\text{-}6)$$

141. Stillwater Barriers. Any structure that restricts the flow of water and causes the water surface to rise above its natural flow profile, would be termed a stillwater barrier. Navigation dams, bridges, weirs, cofferdams, and channel stabilization structures such as wing dams and groins are examples of stillwater barriers. Each of these structures creates an obstruction to the flow of water in the stream, and the effect of the flow constriction is projected upstream by the backwater profile. If the water surface is raised to an abnormal height, the overbank areas would be flooded, thereby creating an effective defensive water barrier. The head necessary to convey a given discharge through a constricted opening is computed by the methods described in Par. 140. If the obstruction is in the form of a weir the head would be computed by the methods described in chapter III.

142. Discharge Rating Curve for Gates Fully Open. The discharge rating of a navigation dam would consist of a series of curves with the upstream water surface elevation plotted against the discharge. Each curve represents a specific number of gates fully open. The general method of computing the discharge rating curve for a single fully opened gate is applicable to any number of gate openings. The discharge rating curve for one gate fully open would be computed as follows:

(1) Assume a discharge and determine the tailwater elevation from the tailwater rating curve.
(2) The head loss through the structure is then computed by the D'Aubuisson or Nagler formula as described in Par. 140.
(3) Add the head loss, computed by equation 7-6, to the tailwater elevation, and this elevation and discharge would form one point on the discharge rating curve.
(4) Other discharges are assumed and the headwater elevations computed and plotted in the same manner as steps (1) through (3). The curve drawn through the points would be the rating curve for one gate completely open.
(5) Other combinations of gate openings would be assumed and the process repeated resulting in a family of curves representing the rating of the structure for the desired gate operating conditions. It must be realized, however, that the discharge from several fully opened gates would not be the same as the product of the number of gates, and the discharge from one gate. The difference in the tailwater elevation and the approach velocity for the increased discharge requires a new computation to be made for each gate schedule.

143. Example. The computation of the discharge rating curve of a low head navigation dam with all of the gates fully open is shown on Plate 706.

SECTION C: CREST GATES PARTIALLY OPEN

144. Applications. The crests of navigation dams, diversion dams that are non-navigable, and run of the river power dams are usually controlled by roller, tainter, or vertical lift gates. When the gates are fully opened the discharge is computed by the method described in Par. 142. The crest gates may be partially opened to release small regulatory flows. The flow under a partially opened gate would either be free or submerged depending on the tailwater elevation for each rate of discharge. If the tailwater is of moderate depth, the jet of water emerging from under the gate may form a hydraulic jump and therefore flow as a free jet under the gate lip. If the tailwater depth is greater than the sequent depth of the hydraulic jump, the jump would be submerged and the flow would act as a submerged jet. Under both conditions the discharge would be computed by means of an orifice type equation. The degree of submergence of the jet by the tailwater or the formation of the hydraulic jump would affect the coefficient of discharge.

145. Tailwater Rating Curve. A tailwater rating curve below the structure would be determined as described in Par. 86.

146. Effective Gate Opening. The effective gate opening on a spillway is defined as the minimum opening between the gate lip and the nearest point to the spillway profile. The gate seat of a spillway gate is normally located downstream from the spillway crest. The effective gate opening on an ogee crest would be less than the normal gate opening because of the curvature of the spillway profile. The effective gate opening for a given opening above the gate seat would be computed as follows:

(1) Draw to scale the spillway crest and gate profile.

(2) Assume various openings of the gate above the gate seat.

(3) Scale the distance from the gate lip to the nearest point on the spillway profile. This minimum opening would be the effective gate opening used in computing the discharge under the gate. The effective and normal gate openings are identical for gates located on horizontal spillway crests.

147. Effective Head. The effective head on a partially opened crest gate would be the head measured from the water surface to the nearest point on the spillway profile as determined in Par. 146.

148. Effective Gate Lip Angle. The effective gate lip angle of a spillway crest gate is defined as the angle between the tangent to the gate lip and the tangent to the spillway profile at the nearest crest point from the gate lip.

149. Effective Gate Length. The effective gate width or length for partial gate openings is normally the net gate length unless there is severe contraction effects due to the pier shape or gate recesses. For conditions of severe contraction effects the gate length would be modified by the same methods as given in Par. 99 for the effective crest length of a spillway.

150. Discharge Capacity. The discharge under a partially opened gate would be computed by the orifice equation

$$Q = C_q bL\,(2gh)^{0.5} \qquad (7\text{-}7)$$

where

Q = the discharge in cfs
b = the effective gate opening in ft.
L = the effective length of gate in ft.
h = the effective head on the gate in ft.
g = acceleration of gravity
C_q = coefficient of discharge

151. Coefficient of Discharge. The coefficient of discharge of submerged vertical sluice gates is a function of the gate shape, the upstream and downstream depths and the gate opening. Plate 707 shows the relationship of the discharge coefficient /5/ as a function of the headwater and tailwater depths and the effective gate opening. This plate is applicable for submerged flow or free discharge.

152. Coefficient of Contraction. The coefficient of contraction of a free or submerged tainter gate or an inclined sluice gate is a function of the effective gate lip angle and the ratio of the head to the gate opening. The relationship between the effective gate lip angle and the coefficient of contraction /6/ for various ratios of head to gate opening is shown on Plates 708 and 709 for the tainter gate and inclined sluice gate respectively.

153. Head Loss. The discharge of a submerged tainter gate is a function of the effective gate lip angle, the gate opening, and the upstream and down stream depths. The effective gate lip angle is defined in Par. 148 and is a function of the radius of the tainter gate arm, the height of the trunnion pin above the gate seat, the shape of the spillway crest and the gate opening. The ratio of the total head loss (H_L) to the tailwater velocity head of a submerged tainter gate /7/ was plotted as a function of the ratio of the tailwater to vena contracta depths for different values of the ratio of headwater to tailwater depths as shown on Plate 710.

154. Discharge Rating Curve for Gates Partially Open. a. The discharge rating of a navigation dam, with gates partially open, would consist of a series of curves with the upstream water surface plotted against the discharge. Each curve represents a fixed condition of gate opening for all of the gates on the dam. Normally each curve represents an operation schedule in which all gates are opened an equal amount. The general method used to compute the discharge rating curve for a single partially opened gate would be applicable for any number of gates if they were all opened an equal amount. The discharge from several partially opened gates, however, would not be the same as the product of the number of gates and the discharge from one gate. The change in tailwater elevation and approach velocity must be considered for each change in gate schedule.

b. Vertical Sluice Gates. The discharge rating curve of a single sluice gate would be computed as follows:

(1) Assume a gate opening and a discharge under the gate.

(2) Compute the head on the gate by Eq. 7-7 with an assumed coefficient of discharge equal to 0.5 as a first approximation.

(3) Determine the tailwater elevation from the tailwater rating curve.

(4) With the upstream and downstream water surface elevations and the gate opening given, determine the coefficient of discharge from Plate 707.

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(5) The discharge is computed by Eq. 7-7 using the discharge coefficient and the head determined in steps (4) and (2). The discharge is compared with the assumed discharge of step (1) and if different a new trial is made until a reasonable balance is secured.

(6) Other discharges are assumed for the same gate opening and the heads computed and plotted in the same manner as given in steps (1) through (5). The curve drawn through the plotted points would be the rating curve for the assumed gate opening.

(7) Other gate openings would be assumed and the process repeated, resulting in a family of curves representing a complete rating of the gate structure.

c. Radial or Tainter Gates. The discharge rating curve of a single tainter gate would be computed as follows:

(1) Assume a discharge and gate opening.

(2) Determine the tailwater depths from the tailwater rating curve for the assumed discharge.

(3) Determine the velocity head $V_2^2/2g$ from the tailwater depth and the discharge per foot of width of gate.

(4) Determine the effective gate opening and the effective gate lip angle as described in Par. 146 and 148 respectively.

(5) Determine the coefficient of contraction from Plate 708, using the effective gate lip angle of (4) above, and an estimated value of the ratio of head to gate opening. (The variation of the contraction coefficient is small for the entire range of ratios of the head to gate opening). If greater accuracy is desired, the contraction coefficient may be revised in a second computation.

(6) Determine the ratio of the tailwater depth to the vena contracta depth. The vena contracta depth would be the product of the contraction coefficient and the effective gate opening determined in steps (4) and (5) above.

(7) Enter Plate 710 with the ratio of tailwater depth to vena contracta depth and determine the value of the ratio of head loss to tailwater velocity head. As in step (5) an average value should be assumed and revised if greater accuracy is desired. Normally for military hydrology purposes an estimate can be made of the ratio of head to tailwater depth and the curve selected that will give a reasonable value of the head loss ratio.

(8) Determine the head loss as the product of the ratio determined in step (7) and the tailwater velocity head determined in step (3) above.

(9) The headwater depth plus the headwater velocity head would be equal to the sum of the tailwater depth, the tailwater velocity head, and the head loss as determined in steps (2), (3), and (8) respectively.

(10) Assuming the velocity head to be zero the headwater depth would then be equal to the value computed in step (9).

(11) Compute the average velocity of approach to the dam as the quotient of the assumed discharge in step (1) and the cross-sectional area of the river upstream from the dam at a depth equal to step (9).

(12) The upstream depth is reduced by an amount equal to the velocity head computed from the average velocity of step (11).

(13) The coefficient of contraction of step (5) and the head loss of step (8) are corrected for the upstream and downstream depths as computed in steps (12) and (1) respectively and steps (1) through (12) repeated if the accuracy warrants.

(14) The discharge is plotted against the upstream water surface elevation and forms one point on the rating curve for the given gate opening.

(15) Other discharges are assumed for the same gate opening and the heads computed and plotted in the same manner as given in steps (1) through (14). The curve drawn through the plotted points would be the rating curve for the assumed gate opening.

(16) Other gate openings would be assumed and the process repeated, resulting in a family of curves representing a complete rating of the gate structure.

(17) The discharges should be checked for the hydraulic jump as described below.

d. Hydraulic Jump. The discharge under a partially opened sluice or tainter gate should be checked to see if the hydraulic jump occurs. If the tailwater depth is at or below the sequent depth for any discharge, the hydraulic jump would be formed. The method of computing the discharge under a tainter gate, as described in (c) above, would be in error if the flow occurred as free efflux with the jump. The discharge under a tainter gate for conditions of free efflux is computed by methods described in Chapter IX. The sequent depth for each discharge is computed as follows:

(1) The initial depth is computed as the product of the contraction coefficient and the effective gate opening.

(2) The unit discharge is determined as the quotient of the discharge and the gate width.

(3) The sequent depth would be determined from Plate 512 for the values of the initial depth and unit discharge of steps (1) and (2) respectively.

e. Roller Gates. A roller gate normally has a lip similar in shape and design to the lip of a tainter gate. Therefore, for partial gate openings the discharge under the lip of a roller gate is computed in the same manner as described for a tainter gate in subparagraph (c) above.

155. Example. The computation of the discharge rating curve of a low head navigation dam with tainter gates partially opened is shown on Plate 711.

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CHANOINE WICKET.

The Chanoine wicket is a narrow wooden leaf which when raised is supported in an inclined position by a prop, and when lowered, lies flat on the foundation just downstream from the sill. The wicket is usually raised and lowered from a maneuver boat. A large number of such wickets side by side constitutes a movable dam.

Bebout wicket on the left, Chanoine wicket on the right. Note the difference between the Bebout & Chanoine sills and foundations.

BEBOUT WICKET.

The Bebout wicket is designed for automatic tripping. The Bebout horse consists of two vertical arms, each hinged at the middle on a horizontal shaft. The diagonal prop is a trussed steel member, pivoted on a shaft fastened to the wicket, and to the downstream end of a base stiffening truss lying on the foundation. The Bebout wicket may be manually tripped, and caused to collapse.

Figure I

CHANOINE & BEBOUT WICKETS

BOULÉ DAM

The Boulé dam consists of a number of collapsible trestles which support movable shutters. Each trestle is a structural steel A-frame placed at right angles to the axis of the dam. Each frame is hinged to the foundation so that the trestles may be lowered behind the sill when the dam is down. The top of each is permanently connected to the top of the next by a chain of greater length than the trestle spacing. When the trestles are erect and the shutters in place the structure forms a dam.

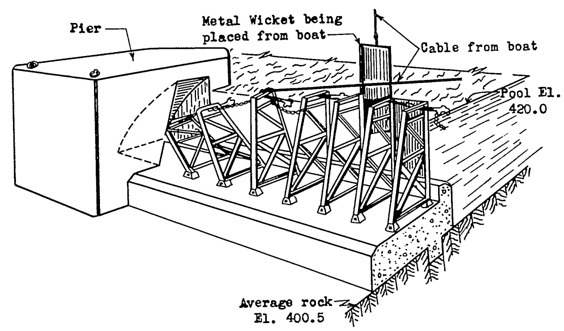

Figure 2
BOULÉ DAM

BEARTRAP GATE

A beartrap gate or weir is built with two sections, or leaves which are hinged at their lower ends. The downstream leaf is a buoyant hollow member. The upstream leaf is a nonbuoyant section fabricated of wood and steel. The beartrap weir forms a flat inverted V in the raised and intermediate positions. The sections are raised by changes in hydrostatic pressure, introduced through valve controlled culverts on the underside of the leaves.

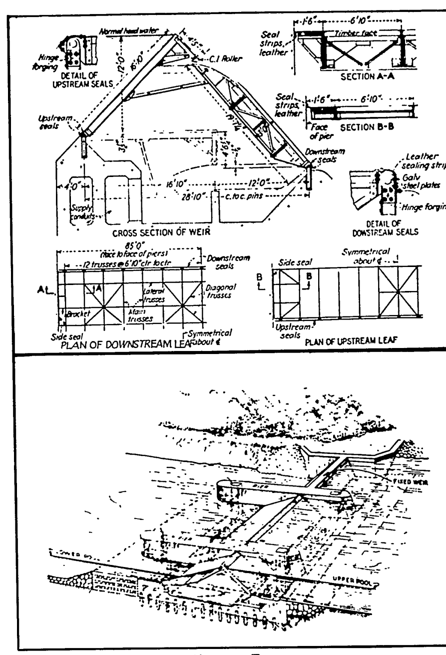

Figure 3
BEARTRAP GATE

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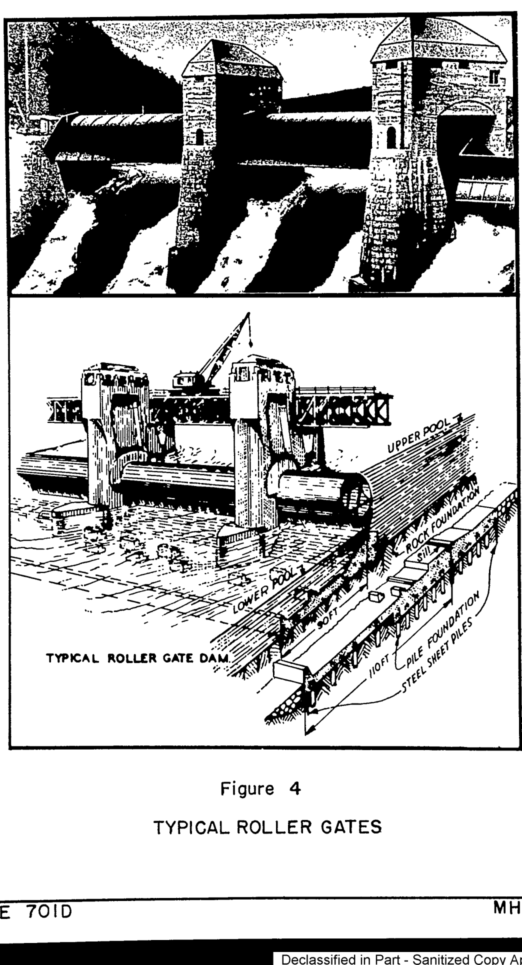
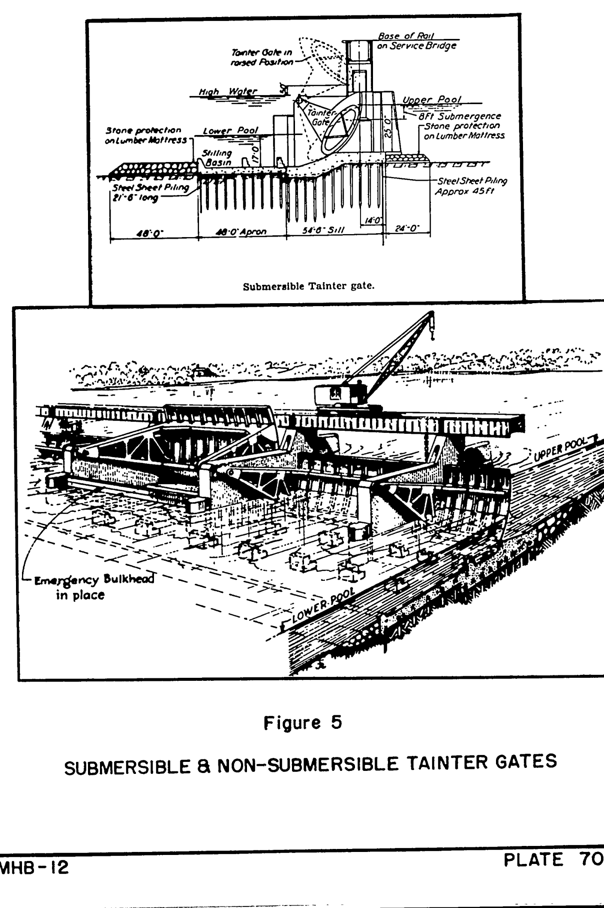


Figure 4

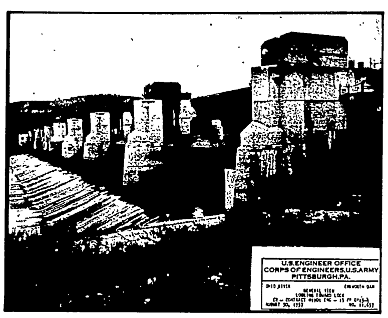
TYPICAL ROLLER GATES



Submersible Tainter gate.

Figure 5

SUBMERSIBLE & NON-SUBMERSIBLE TAINTER GATES



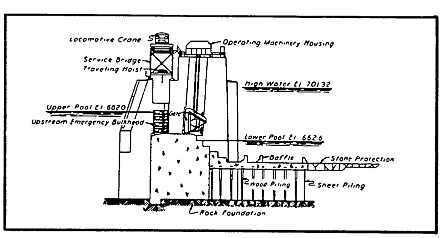
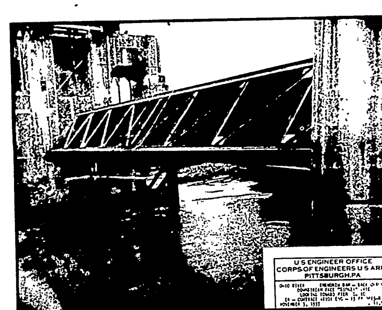


Figure 6

VERTICAL LIFT GATES



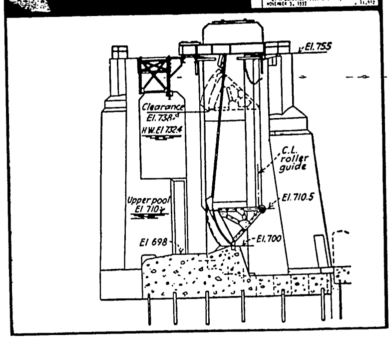


Figure 7

SIDNEY GATE

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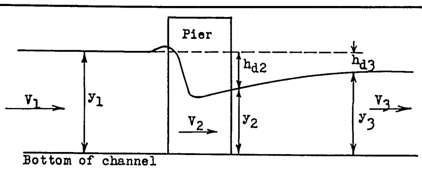
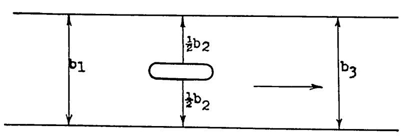
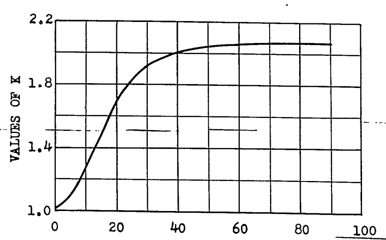


PROFILE



PLAN



PERCENTAGE OF CHANNEL OBSTRUCTED BY PIER

PIER DIAGRAM & K COEFFICIENTS

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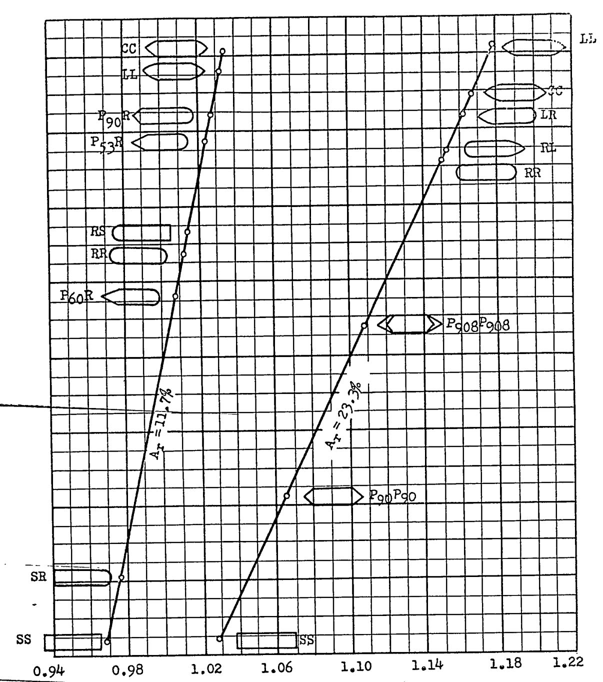


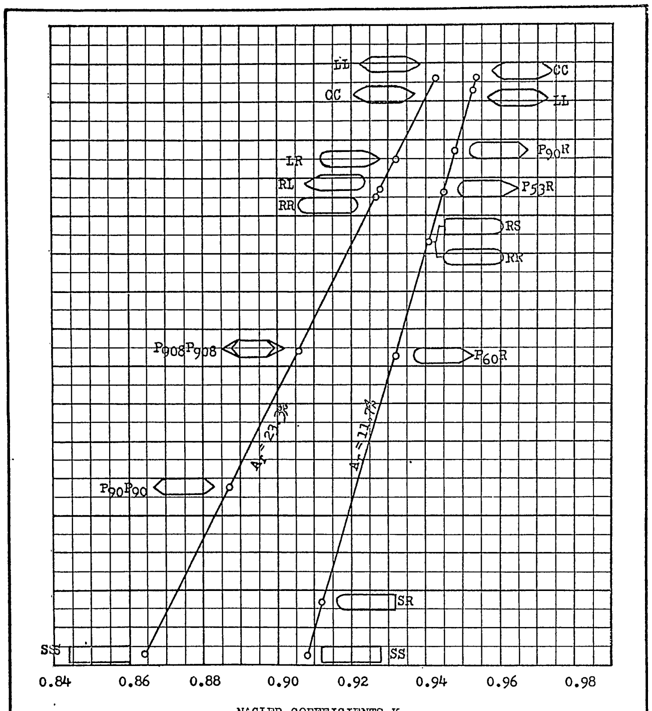
D'AUBUISSON COEFFICIENTS K_D

NOTE:

D'Aubuisson coefficients for various shaped piers, pier length equal to 4 times pier width, and channel contractions of 11.7 and 23.3 percent.

Flow is Class 1.

D'AUBUISSON COEFFICIENTS FOR VARIOUS PIER SHAPES



NOTE:

Nagler Coefficients for various shaped piers, pier length equal to 4 times pier width, and channel contractions of 11.7 and 23.3 percent.

Flow is Class 1.

NAGLER COEFFICIENT FOR VARIOUS PIER SHAPES

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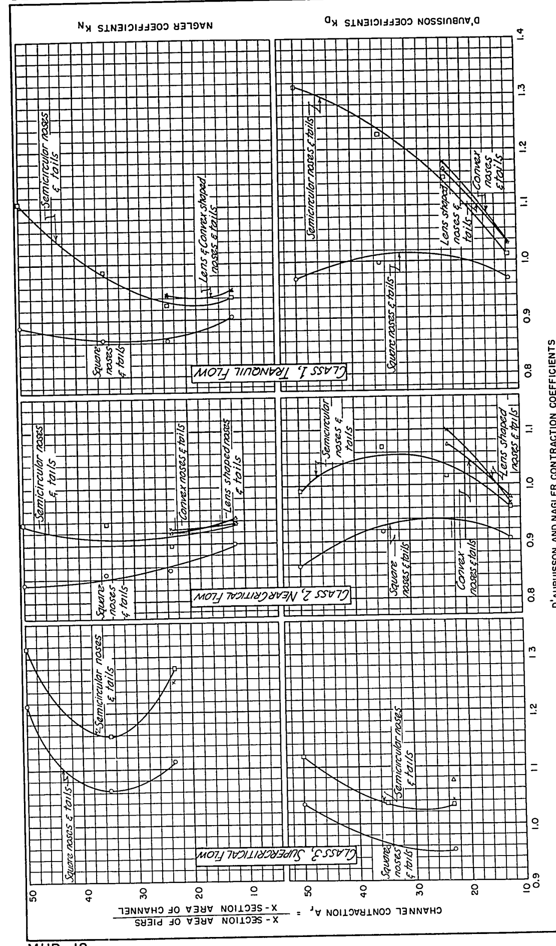
D'AUBUISSON & NAGLER PIER LOSS COEFFICIENTS



DETERMINATION OF THE DISCHARGE RATING CURVE FOR A LOW HEAD NAVIGATION DAM

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(5) Assumed physical data of a low head navigation dam.

(6) The tailwater rating curve was computed by methods given in Chapter V and plotted as shown.

HEAD DISCHARGE COMPUTATIONS
TAINTER GATES FULLY OPEN

(7) The discharge a low head navigation dam with all gates fully open was computed as described in the following steps:

1. The discharges were assumed as tabulated in Col. 1, Table I.

2. The tailwater elevation for each discharge listed in Col. 1 was determined from the tailwater rating curve and entered in Col. 2.

3. The average velocity through the most contracted section of the dam was computed for each discharge. The flow area was taken as the product of the net gate opening and the tailwater depth. Therefore the average velocity was the quotient of the discharges of Col. 1 divided by 1000 times the tailwater depth of Col. 2, and entered in Col. 3.

4. The velocity heads of the average velocities of Col. 3 were determined and entered in Col. 4.

5. The head loss was computed by equation 7-6. The value of K_D = 1.22 was taken from Plate 705 for piers with semicircular noses and tails and a channel contraction ratio of 1- 1000/1500 = 33% for class 1 flow. The equation of the head loss is:

$$H_L + V_1^2/2g = (\frac{1}{1.22})^2 V_2^2/2g + 0.15 V_2^2/2g$$

$$H_L = V_1^2/2g = 0.82 V_2^2/2g$$

6. Values of the velocity head at the contracted section listed in Col. 4 were multiplied by the coefficient 0.82 and entered in Col. 5.

7. The total head H including the approach velocity head was computed by adding the tailwater depth of Col. 2, the velocity head of the contracted area of Col. 4, and the head loss plus approach velocity head of Col. 5, and entered in Col. 6.

8. The approach velocity head was computed by dividing the discharges of Col. 1 by 1500 ft., the upstream channel width, to get the discharge per foot of width. The discharge per foot of width was then divided by the total upstream head given in Col. 6 to give the average approach velocity. The approach velocity head was determined from the average approach velocity and entered in Col. 7.

9. The velocity heads of Col. 7 were subtracted from the total heads of Col. 6 to give the upstream depth, and were listed in Col. 8.

10. The average approach velocity head of Col. 7 was corrected by using the upstream depths in Col. 8 instead of the total heads of Col. 6 as described above and the corrected depth was entered in Col. 9.

DISCHARGE RATING CURVE

(8) The headwater depths listed in Col. 9, Table I were plotted against the discharges of Col. 1 and the discharge rating curve drawn.

DETERMINATION OF THE DISCHARGE RATING CURVE FOR A LOW HEAD NAVIGATION DAM

ALL GATES FULLY OPEN

Item

INITIAL DATA

(1) 20 - 50' x 20' Tainter gates for a low head dam (gates fully open)

(2) Gate seats on a flat sill at river bottom elevation

(3) Piers 8' wide with semi-circular noses and tails

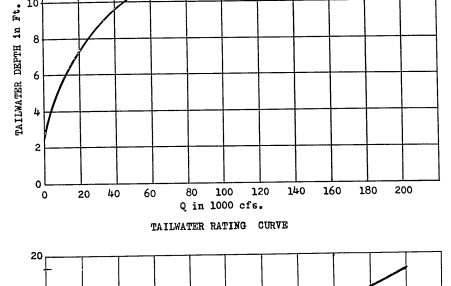
(4) Piers parallel to the current

(5) Total width of river above dam = 1500 ft.

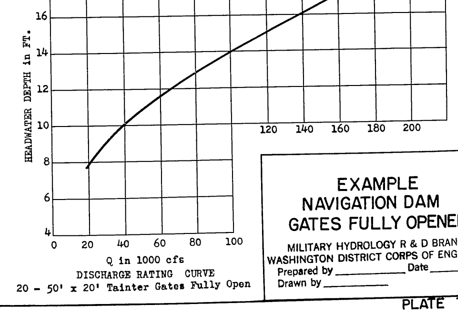
(6) Tailwater rating curve at the dam is shown

(7) TABLE I
HEAD DISCHARGE COMPUTATIONS

Q cfs Col. 1	y_2 ft. Col. 2	V_2 ft/sec Col. 3	$V_2^2/2g$ ft. Col. 4	$H_L + V_1^2/2g = 0.82V_2^2/2g$ Col. 5	H = ② + ④ + ⑤ ft. Col. 6	$V_1^2/2g$ ft/sec Col. 7	y_1 ft. Col. 8	Corrected y_1 Col. 9
20,000	7.6	2.63	0.107	0.09	7.80	0.04	7.76	7.76
60,000	11.0	5.46	0.462	0.384	11.84	0.18	11.66	11.66
100,000	12.4	8.06	1.01	0.83	14.24	0.34	13.90	13.88
140,000	13.3	10.5	1.72	1.41	16.43	0.50	15.93	15.90
200,000	14.0	14.3	3.16	2.60	19.76	0.71	19.05	19.00



TAILWATER RATING CURVE



DISCHARGE RATING CURVE
20 - 50' x 20' Tainter Gates Fully Open

EXAMPLE
NAVIGATION DAM
GATES FULLY OPENED

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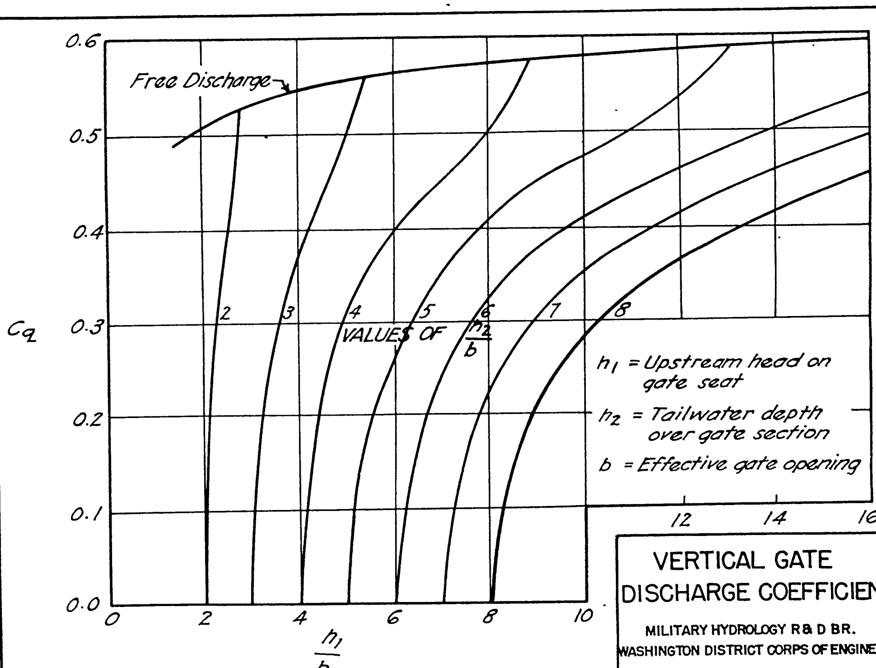

VERTICAL GATE DISCHARGE COEFFICIENT

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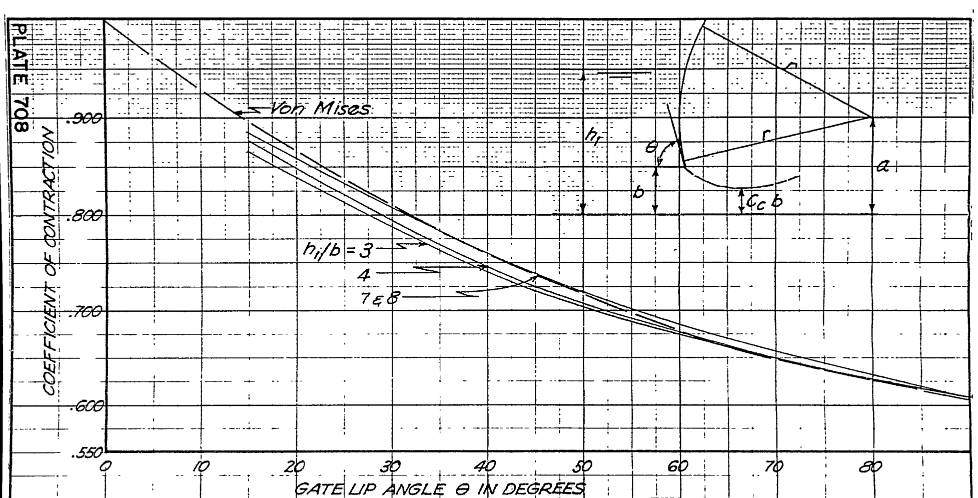
PLATE 708



COEFFICIENT OF CONTRACTION
TAINTER GATE

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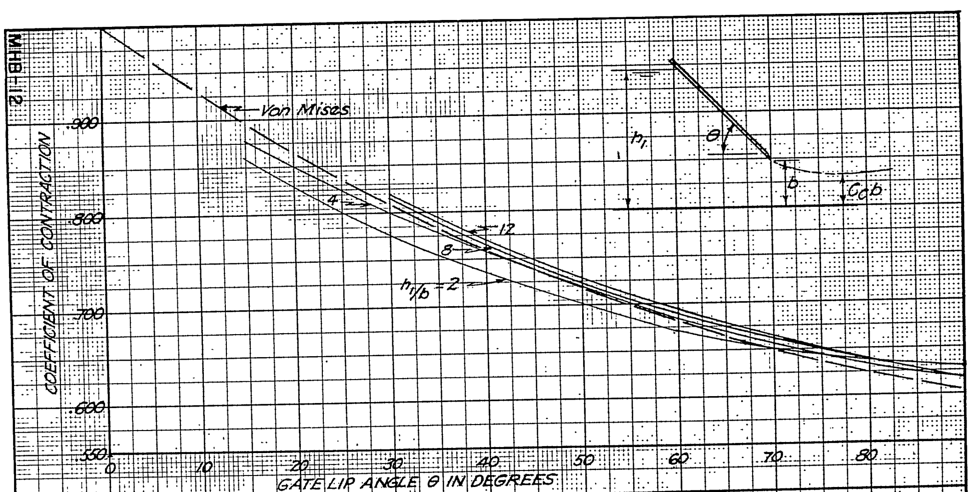
MHB - 12

MHB - 12



COEFFICIENT OF CONTRACTION
INCLINED GATE

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PLATE 709

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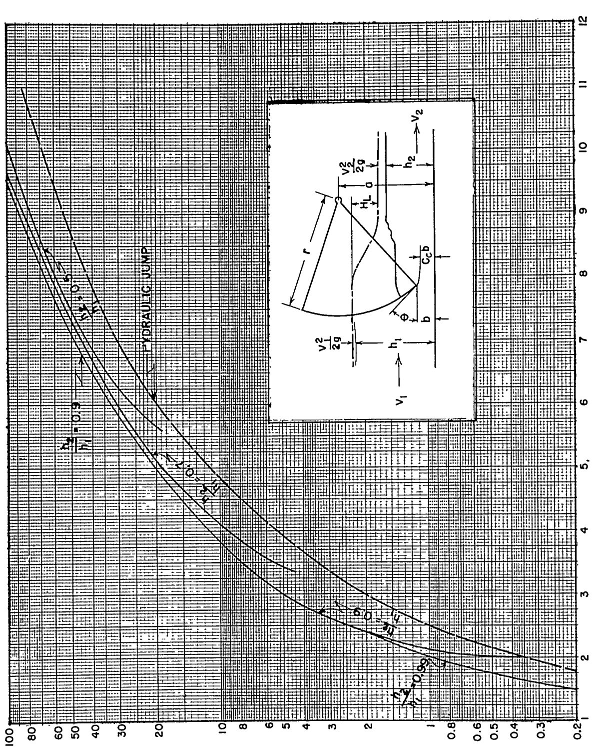

HEADLOSS
SUBMERGED TAINTER GATE

DETERMINATION OF THE DISCHARGE RATING CURVE
FOR SUBMERGED TAINTER GATES

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(7) The physical data was given in items (1) through (7).

(8) The tailwater rating curve was given on Plate 706.

GATE ANGLE AND CONTRACTION COEFFICIENT

(9) The gate lip angle and the coefficient of contraction were determined as described in the following steps:

1. The gate openings were assumed as shown in Col. 1, Table I.

2. The cosine of the gate lip angle was computed for each value of the gate opening in Col. 1 and entered in Col. 2. An example for a gate opening of 2.0 feet is as follows:

$$\text{Cos } \theta = \frac{\text{(Trunnion Pin Elev. - Gate Lip Elev.)}}{\text{(Radius of Curvature of the Tainter Gate)}}$$

$$= \frac{20 - 2}{30} = 0.600$$

3. The arc-cosine of the tabulated values in Col. 2 were determined and tabulated in Col. 3.

$$\theta = \text{Cos}^{-1}\ 0.600 = 53.1^{\circ}$$

4. The coefficient of contraction was determined from Plate 708 for each of the values of θ listed in Col. 3 and tabulated in Col. 4.

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HEAD-DISCHARGE COMPUTATIONS
PARTIALLY OPENED TAINER GATES

(10) The head and discharge of the partially opened tainter gates was computed as described in the following steps:

1. Discharges were assumed for each gate opening as shown in Col. 1.

2. The discharge per foot of width passing under the gates was computed by dividing the discharges in Col. 1 by the effective gate width of 1000 feet and was entered in Col. 2.

3. The tailwater depth was determined from the tailwater rating curve for the discharges given in Col. 1 and tabulated in Col. 3.

4. The average velocity per foot of width was computed by dividing the values of Col. 2 by Col. 3 and entering in Col. 4.

5. The velocity head was determined from Col. 4 and listed in Col. 5.

6. The value of C_cb was determined for each gate opening from the values of C_c given in Col. 4 of Table I.

7. The ratios of h_2/C_cb were determined for each tailwater depth of Col. 3 and the respective values of C_cb and tabulated in Col. 6.

8. For each value of the ratio h_2/C_cb in Col. 6 the value of $H_L/V_2^2/2g$ was determined from Plate 710 and entered in Col. 7.

9. The product of velocity heads in Col. 5 and the ratios of $H_L/V_2^2/2g$ gave the head loss H_L and was listed in Col. 8.

10. The total head H was determined as the sum of the tailwater depth and velocity head plus the head loss H_L and was entered in Col. 9.

11. The upstream water surface depth was determined as the difference between the total head "H" of Col. 9 and the upstream velocity head. The discharge per foot of width was computed by dividing the discharge in Col. 1 by 1500 ft., the width of approach channel upstream from the dam. The average approach velocity was determined, as a first approximation, by dividing the unit approach discharge by the total head given in Col. 9. The velocity head for the unit approach discharge was computed and subtracted from the total head H and listed in Col. 10. Normally the approach velocity is so small that the difference in the velocity head is negligible whether computed from a flow depth equal to the total head, or a flow depth equal to the actual head. For high approach velocity heads a second check should be made using the computed depth h_1, to check the approach velocity head computed by using the depth H.

DISCHARGE RATING CURVE

(11) A discharge rating curve for each of the five partial gate openings was plotted from the values of Col. 1 and Col. 10 and shown on Plate 711 C. Each curve represents a gate operation schedule in which all 20 of the gates were opened an equal amount.

DETERMINATION OF DISCHARGE RATING CURVE FOR SUBMERGED TAINTER GATES

Item

INITIAL DATA

(1) 20 - 50' x 20' Tainter gates for a low head dam.

(2) Gate seat on a flat sill at river bottom elevation.

(3) Gate arm and radius of curvature of tainter gate = 30'

(4) Trunnion pin height above the gate seat = 20'

(5) Piers 8' wide and 50' long below gate seat.

(6) Pier shape gives negligible contraction at partial gate openings.

(7) Total width of river above dam = 1500 ft.

(8) Tailwater rating curve at the dam is shown on Plate 7205.

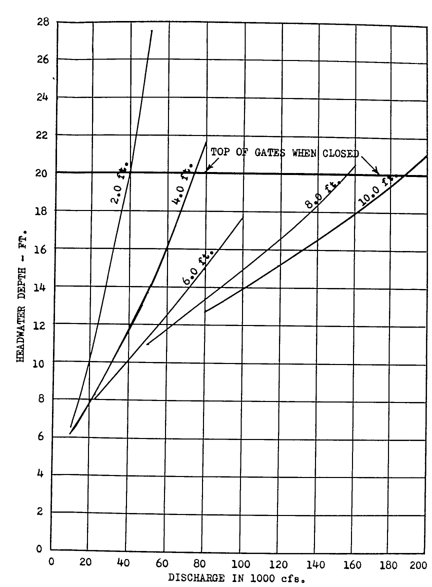
(9) TABLE I
GATE ANGLE 0 AND CONTRACTION COEFFICIENT

Gate Opening ft. Col. 1	$\frac{a-b}{r}$ ft. Col. 2	$0 = \cos^{-1}\frac{a-b}{r}$ ft. Col. 3	C_c Col. 4
0	0.666	48.2°	0.72
2	0.600	53.1°	0.70
4	0.533	57.8°	0.69
6	0.466	62.2°	0.68
8	0.400	66.4°	0.66
10	0.333	70.6°	0.65

Item

(10) TABLE II
HEAD DISCHARGE COMPUTATIONS

Q cfs Col. 1	q cfs/ft Col. 2	h_2 ft Col. 3	V_2 ft/sec Col. 4	$V_2^2/2g$ ft Col. 5	h_2/C_cb Col. 6	$H_L/V_2^2/2g$ (Pl. 7304) Col. 7	H_L ft Col. 8	H ft Col. 9	h_1 ft Col. 10
Gate Opening = 2.0'					C_cb = 1.44 ft.				
10,000	10.0	6.0	1.67	0.043	4.16	11.6	0.5	6.5	6.5
15,500	15.5	7.0	2.21	0.076	4.86	17.1	1.3	8.4	8.4
22,500	22.5	8.0	2.81	0.123	5.56	24.3	2.99	11.11	11.11
40,000	40.0	9.7	4.13	0.26	6.73	39.5	10.3	20.26	20.2
50,000	50.0	10.4	4.80	0.36	7.22	47.0	16.9	27.6	27.5
Gate Opening = 4.0'					C_cb = 2.80 ft.				
10,000	10.0	6.0	1.67	0.042	2.14	1.34	0.05	6.09	6.09
22,500	22.5	8.0	2.81	0.123	2.85	3.50	0.43	8.5	8.5
50,000	50.0	10.4	4.80	0.36	3.71	8.50	3.06	13.8	13.7
80,000	80.0	11.8	6.78	0.71	4.22	12.2	8.67	21.8	21.7
Gate Opening = 6.0'					C_cb = 4.08 ft.				
22,500	22.5	8.0	2.81	0.123	1.96	0.90	0.11	8.23	8.2
50,000	50.0	10.4	4.80	0.36	2.55	2.50	0.90	11.66	11.5
80,000	80.0	11.8	6.78	0.71	2.89	3.85	2.73	15.24	14.8
100,000	100.0	12.4	8.06	1.01	3.04	4.6	4.65	18.0	17.8
Gate Opening = 8.0'					C_cb = 5.36 ft.				
50,000	50.0	10.4	4.80	0.36	1.94	0.86	0.31	11.07	11.0
80,000	80.0	11.8	6.78	0.71	2.20	1.50	1.06	13.57	13.3
100,000	100.0	12.4	8.06	1.01	2.31	1.80	1.82	15.23	14.9
125,000	125.0	13.0	9.61	1.44	2.42	2.10	3.02	17.46	17.1
160,000	160.0	13.6	11.80	2.15	2.54	2.45	5.26	21.01	20.6
Gate Opening = 10.0'					C_cb = 6.50 ft.				
80,000	80.0	11.8	6.78	0.71	1.82	0.60	0.43	12.94	12.7
100,000	100.0	12.4	8.06	1.01	1.91	0.80	0.81	14.22	13.9
125,000	125.0	13.0	9.61	1.44	2.00	1.00	1.44	15.88	15.45
160,000	160.0	13.6	11.80	2.15	2.09	1.20	2.58	18.33	17.8
200,000	200.0	14.0	14.3	3.16	2.16	1.40	4.43	21.59	21.0



DISCHARGE RATING CURVE
PARTIAL GATE OPENING
20 - 50' x 20' Tainter Gates

EXAMPLE
NAVIGATION DAM
GATES PARTIALLY OPENED

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WASHINGTON DISTRICT CORPS OF ENGINEERS
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Drawn by ____________

CHAPTER VIII
RESERVOIR OUTLET CONDUITS

SECTION A: BASIC CONSIDERATIONS

157. Function. a. Reservoir outlet conduits are designed to function as flood control outlets, navigation control regulators, power penstocks, irrigation flow release structures, water supply intakes, and any combination of the above for use as a multiple purpose outlet.

b. The outlet works for concrete and masonry dams are usually conduits or sluices through the structure, and are shorter than those required for earth dams. The conduits may be controlled by gates at the upstream face or by valves from a gallery in the interior of the dam. The conduits may be located at several levels to reduce the pressure head on the gates. Conduits are generally spaced in the overflow section of the dam so that they discharge directly into the spillway stilling basin. If the conduits are carried through the non-overflow sections, a separate stilling basin is usually provided.

c. For earth or rockfill dams the base width of the embankment determines the length of conduits. A hydraulic fill earth dam usually requires the longest conduits, the rolled fill earth dam requires slightly shorter conduits, and the rock filled dam the shortest conduits of the three.

158. Types of Outlet Conduits. a. Masonry dams usually have a large number of small conduits for flexibility of operations while earth fill dams have a small number of large conduits for economy of construction. The most common types of outlet conduits can be classed as flood control conduits and regulating sluices.

b. Flood Control Conduits. Flood control conduits are designed for large capacities, usually about equal to the bankfull capacity of the river channel downstream from the dam.

c. Regulating Sluices. Regulating sluices are a type of conduit that gives close control for the release of water. The various types of regulating sluices are listed as follows:

(1) Outlets used to regulate the flow for navigation purposes are required to operate continuously over long periods of time, and are normally of lower capacity than flood control conduits.

(2) Power penstocks are normally designed as steel lined conduits which conduct the flow to hydraulic turbines, with relatively low velocities at the design discharge.

(3) Irrigation flow release conduits are designed to give close regulation and conservation of water. The irrigation outlet sometimes discharges into a canal at a higher elevation than the river bed.

(4) Municipal water supply intakes are often designed as multiple outlets to assure reliability and to enable the water to be drawn from various heads for temperature and algae control. The case of maintenance without interruption of service is of primary importance.

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159. Component Parts of Outlet Works. The component parts of outlet works very with each type of dam, but generally consist of an intake structure, conduits, and a stilling basin. The intake structure normally consists of trash racks, a shaped entrance, a transition section, and a gate chamber for the emergency and service gates with air vents for each gate.

160. Pressure Gradient at Exit Portal. In pressure flow computations /1/ when the momentum of the water issuing from a conduit is sufficient to push the tailwater away from the outlet, the pressure gradient would intersect near the center of the outlet portal of a circular conduit. The pressure gradient for a rectangular conduit would intersect the outlet portal at a point approximately 2/3 the height of the conduit.

SECTION B: BASIC HYDRAULIC THEORY

161. Considerations. a. The hydraulic analysis of flow through an outlet conduit or sluice involves consideration of two conditions of flow:

(1) When the reservoir water surface elevation is at low stages so that the water flows with a free water surface as an open channel.

(2) When the stage increases to fill the outlet conduit so that it flows under pressure.

b. In a normal reservoir more than half of the volume of storage is in the top third of the pool depth. The rate of discharge from an outlet conduit is a function of the head on the conduit and therefore greatest at a full reservoir. The upper portion of the discharge rating curve is of particular importance in military hydrology because it offers the best opportunity for creating artificial floods by outlet regulation.

162. Discharge Capacity. a. Pressure Flow. The discharge capacity of an outlet conduit would be computed by means of the basic pipe flow equation as described in paragraph 44, when the conduit flows under pressure.

b. Open Channel Flow. The discharge capacity for low flows is dependent on whether the tailwater elevation controls the flow, or whether the reservoir water surface controls the discharge.

(1) Tailwater Control. For very low flows, when the difference between the reservoir water surface and tailwater elevation is small, the tailwater elevation would probably control the rate of discharge. The discharge capacity is determined by assuming a discharge and computing the water surface profile through the conduit. The starting elevation of the water surface profile would be determined from the tailwater rating curve for the assumed discharge. All losses would be considered such as the gate recess loss, entrance loss, friction loss, etc.

(2) Headwater Control. When the reservoir water surface rises sufficiently to overcome the tailwater effects, but not cause the conduit to flow under pressure, the control would move upstream to the conduit entrance. Under this condition, the discharge capacity would be computed by the formula:

$$y_1 = y_c + K_e \frac{Vc^2}{2g} + \frac{Vc^2}{2g} \qquad (8\text{-}1)$$

where

y_1 = the upstream depth in feet above the invert of the conduit at the entrance
y_c = critical depth in the conduit in feet
$\frac{Vc^2}{2g}$ = velocity head in the conduit at the entrance section
K_e = entrance loss coefficient.

163. Total Head. a. The total head on a conduit flowing under pressure is consumed in overcoming intake, gate, bend and friction losses, and also in producing the discharge velocity head. This is expressed in the following equation:

$$H = h_e + h_g + h_b + h_f + h_o \qquad (8\text{-}2)$$

where

H = the total head in feet
h_e = entrance head loss in feet
h_g = gate head loss in feet
h_b = bend loss in feet
h_f = head loss due to friction in feet
h_o = outlet velocity head

b. The total head, for a conduit flowing under pressure with the gates completely open, would be added to the elevation where the pressure gradient intersects the downstream portal. The losses are expressed as a function of the velocity head in the conduit, giving the equation:

$$H = (K_e + K_g + K_b + K_f + 1) \frac{V^2}{2g} \qquad (8\text{-}3)$$

where

K = velocity head coefficient for entrance, gate, bend, and friction losses respectively

$\frac{V^2}{2g}$ = velocity head in feet at the outlet.

164. Intake and Gate Loss. A combined coefficient ($K_e + K_g$) for entrance, gate and transition losses has been determined from model and prototype data /1/ and are shown on Plate 801. These coefficients are conservative but represent well-designed flood control conduits. If the intake water surface is low and the water surface drops through critical depth, the loss factor in Plate 801 should be doubled.

165. Friction Loss. Friction loss would be computed by the use of Manning's formula converted for use as a coefficient of the velocity head.

$$h_f = K_f \frac{V_o^2}{2g} \tag{8-4}$$

$$K_f = \frac{29.14\ n^2 L}{R^{4/3}} = C_f \frac{L}{R^{4/3}} \tag{8-5}$$

C_f for various values of "n" are given in paragraph 48.

166. Bend Loss. a. The energy loss involved in the flow around a bend may be expressed by the equation:

$$h_L = h_f + h_b \tag{8-6}$$

where

h_L = the aggregate head loss
h_f = the friction loss of an equal length of straight conduit
h_b = the bend loss

b. The energy loss h_b is caused by internal friction due to the disruption of the velocity pattern in the bend. The magnitude of the energy loss is a function of the conduit shape, the degree of curvature of the bend, the velocity of flow, and the size of the conduit.

c. The bend loss is expressed as a function of the velocity head:

$$h_b = K_b \frac{V^2}{2g} \tag{8-7}$$

in which K_b is a function of the following factors:

$$(r/D,\ \theta,\ N_r,\ y/b) \tag{8-8}$$

where

K_b = bend loss coefficient
r = radius of bend
D = diameter of conduit
θ = deflection angle of bend

N_r = Reynolds number
y/b = ratio of conduit depth to breadth in rectangular conduits and termed the aspect ratio.

d. The Reynolds number need not be considered for ordinary design or analysis purposes but it would be of importance in model studies. The effect /2/ of the ratio r/D and deflection angle θ on the bend coefficient is shown on Plate 802.

e. A correction factor as a function of the aspect ratio would be applied to the above bend coefficient for rectangular conduits. Plate 803 shows /3/ the relation between the correction factor as a function of the aspect ratio, and r/D ratio.

f. A problem showing the computation of the bend loss for a rectangular sluice is given on Plate 804.

167. Discharge Rating Curve. The discharge rating curve of an outlet conduit consists of an upper and lower segment. The lower segment of the rating curve is controlled by the conduit entrance or the tailwater elevation when the reservoir water surface is at low stages. The water, under this condition, flows through the conduit with a free surface as an open channel. The upper portion of the rating curve is controlled by the capacity of the conduit when flowing full under pressure.

a. Open Channel Flow. The lower segment of the discharge rating curve of an outlet conduit, is determined by open channel methods described in Chapter V. When the reservoir pool elevation is low the control section is either at the conduit outlet, due to tailwater conditions; or at the conduit entrance, due to the critical flow conditions at the entrance. The discharge rating curve is computed as follows:

(1) Several discharges are assumed and the tailwater elevations determined from the tailwater rating curve.

(2) The water surface profiles are determined for each of the discharges in the same manner as given on Plate 515.

(3) The lower segment of the discharge rating curve is plotted from the resulting reservoir elevations and the assumed discharges.

b. Pressure Flow. The upper portion of the discharge rating curve would be computed by equation 8-3 as follows:

(1) Determine the velocity head coefficients by the methods described in paragraph 164 to 166, inclusive.

(2) Add the coefficients and substitute in the equation:

$$H = (\Sigma K) \frac{V^2}{2g}$$

(3) Transform the equation to $Q = AV = A\ (2gH/\Sigma K)^{0.5}$.

(4) Assume values of the total head and compute the discharge. The total head is measured from the center of the outlet portal to the reservoir water surface.

(5) Plot the reservoir elevation against the discharge and draw the rating curve.

168. Example. The computation of the discharge rating curve of a flood control sluice for a large dam is shown on Plate 805.

169. References.

/1/ Preliminary Draft, "Part CXVI Hydraulic Design, Chapter 2, Reservoir Outlet Structures". Engineering Manual for Civil Works, Office Chief of Engineers, Corps of Engs., Dept. of the Army.

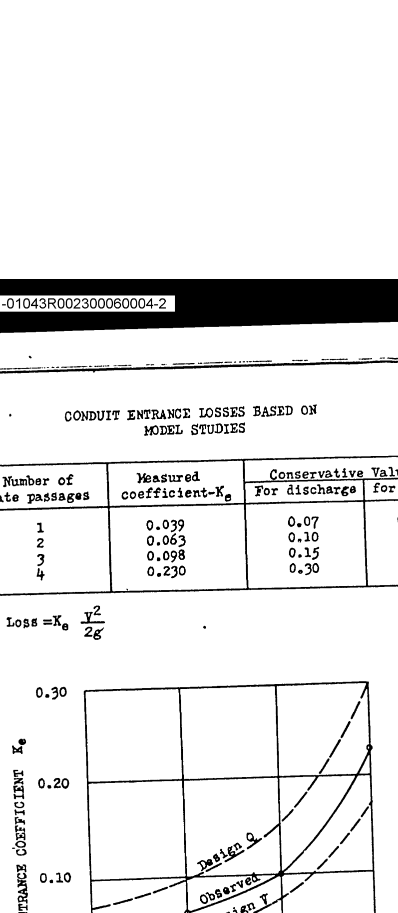
/2/ Wasielewski, R. "Verluste in Glatlen Rohrkrummern mit Krusrudem Querschnitte Bei Weniger als 90° Ablenkung" ("Loss in Smooth Pipe Bends of Circular Cross Section for Deflections of less than 90°"). Mitterlungen Das Hydraulischen Institutes Der Technischen Hochschule. Vol. 5, pp 53-67, 1952.

/3/ Fan Engineering. Buffalo Forge Co., 1938.

CONDUIT ENTRANCE LOSSES BASED ON MODEL STUDIES

Model study	Number of gate passages	Measured coefficient-K_e	Conservative Values	
			For discharge	for velocity
Wolf Creek	1	0.039	0.07	0.020
Dennison	2	0.063	0.10	0.035
Arkabutla	3	0.098	0.15	0.070
Sardis	4	0.230	0.30	0.180

h_e = Entrance Loss = $K_e \frac{V^2}{2g}$



ENTRANCE LOSS COEFFICIENTS

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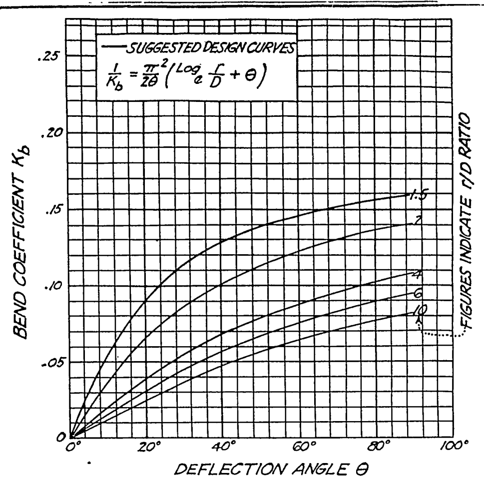


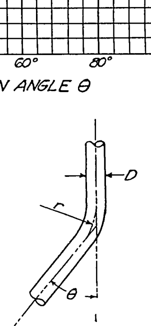
BASIC EQUATION = $h_b = K_b \frac{V^2}{2g}$

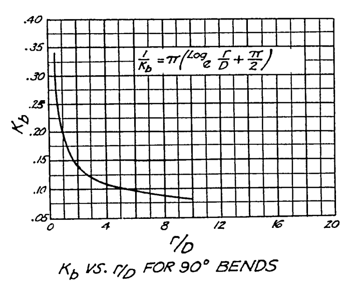
h_b = HEAD LOSS DUE TO BEND

K_b = BEND LOSS COEFFICIENT

V = VELOCITY IN PIPE

FIGURES ON GRAPH INDICATE r/D RATIO





K_b VS. r/D FOR 90° BENDS

BEND LOSS COEFFICIENTS

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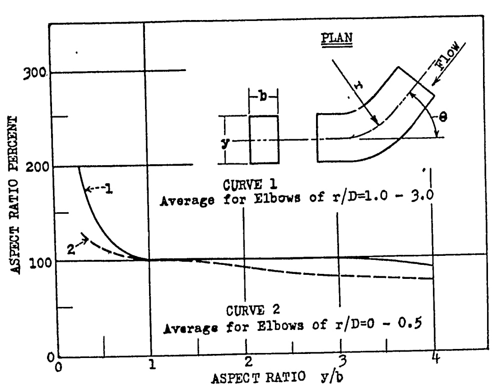

Pressure Loss in Per Cent of an Aspect Ratio of Unity

ASPECT RATIO

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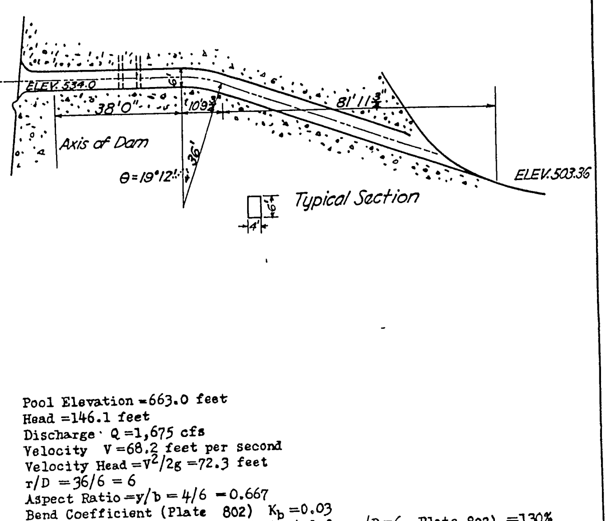

Pool Elevation = 663.0 feet
Head = 146.1 feet
Discharge $Q = 1,675$ cfs
Velocity $V = 68.2$ feet per second
Velocity Head $= V^2/2g = 72.3$ feet
$r/D = 36/6 = 6$
Aspect Ratio $= y/b = 4/6 = 0.667$
Bend Coefficient (Plate 802) $K_b = 0.03$
Aspect Ratio Percent Factor Estimated for $r/D = 6$, Plate 803) = 130%
Corrected Bend Coefficient $K_b = 1.3 \times 0.03 = 0.04$

EXAMPLE BEND LOSS

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DETERMINATION OF THE DISCHARGE RATING CURVE OF A RESERVOIR OUTLET CONDUIT

EXPLANATION OF COMPUTATIONS

<u>Item</u>

INITIAL DATA

(1)-(7) Assumed physical data for a large flood control conduit of a dam.

COMPUTATION OF WATER SURFACE PROFILE

(8) The water surface profile was computed up the circular conduit in the same manner as described on Plate 515. The initial water surface elevation was determined from the tailwater rating curve with a total discharge of 50,000 cfs. The computation of the water surface profile in the conduit was determined for a discharge of 5,000 cfs.

(9) The depth of flow as determined from the water surface profile of item (8) equaled the critical depth at station 4 + 38. Since the water surface profile approached the critical depth within the conduit it indicated that the conduit was constructed on a slope greater than critical for a discharge of 5,000 cfs. The water surface profile was of similar type to the example given on Plate 515, with the control at the conduit entrance. The reservoir water surface elevation was determined from Eq. 8-1 as described in the following steps:

1. The invert elevation at station 0 + 50 was determined from the bottom slope, conduit length, and invert elevation at the outlet portal.

2. The critical depth was computed as described in Par. 72 and given in item (5).

3. The velocity head was determined from the discharge and the cross sectional area at a depth $y_c = 14.0$ ft.

4. The combined entrance loss (entrance, gate, and transition losses) was computed as described in Par. 164. The loss coefficient was determined from the design discharge curve on Plate 801 for two gate passages in the conduit. The entrance coefficient was doubled as the water surface dropped through the critical depth as stated in Par. 164.

DISCHARGE COMPUTATIONS FOR PRESSURE FLOW

(10) The pressure flow discharge rating curve was determined from Eq. 8-3 as described in the following steps:

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1. The reservoir water surface elevations were assumed and listed in Col. 1, Table II.

2. The total head was measured from the center of the outlet portal at elevation 1230 ft. msl. The total heads were determined as the difference between the reservoir water surface elevations and 1230 ft. msl., and entered in Col. 2.

3. The discharge as entered in Col. 4 was determined from the head as follows:

$$H = (K_e + K_f + 1.0)\ V^2/2g \qquad \text{(Eq. 8-3)}$$

$K_e = 0.1$ Plate 801 for 2 gate passages and design Q curve)

$$K_f = C_f L/R^{4/3} \qquad \text{(Eq. 8-5)}$$

$$= (0.00492)\ (820)/5.5^{4/3}$$

$$= 0.42$$

$$H = (0.1 + 0.42 + 1.0)\ V^2/2g = 1.52\ V^2/2g$$

$$V = (2gH/1.52)^{0.5} = 6.51\ H^{0.5}$$

$$Q = AV = (\pi\ 22.0^2/4)\ (6.51\ H^{0.5}) = 2475\ H^{0.5}$$

(11) The discharge rating curve was plotted from the elevations and discharges of item (9), and from Col. 1 and 4, Table II.

TABLE I
COMPUTATION OF WATER SURFACE PROFILE

(8)

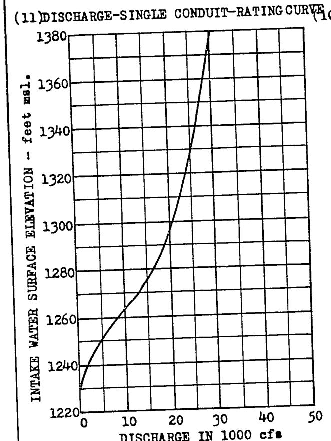
Station Col. 1	Water Surf. Elev. ft. msl Col. 2	Bottom Elev. ft. msl. Col. 3	Depth feet Col. 4	Area $feet^2$ Col. 5	Velocity ft/sec Col. 6	$\frac{V^2}{2g}$ feet Col. 7	ΔH_o feet Col. 8	$R^{2/3}$ Col. 9	Slope S Col. 10	So- S Col. 11	ΔL feet Col. 12
8+70	1240.6	1219.0	21.6	378	13.23	2.72		3.27			
			+0.6		13.315	-0.07	+0.53	3.325	.001232	.010268	52
8+18			21.0	373	13.40	2.79		3.38			
			+2.0		13.86	-0.40	+1.60	3.46	.001230	.01027	156
6+62			19.0	349	14.32	3.19		3.54			
			+2.0		15.10	-0.73	+1.27	3.545	.001391	.010109	125
5+37			17.0	315	15.88	3.92		3.55			
			+2.0		16.99	-1.17	+0.83	3.51	.001797	.009703	86
4+51			15.0	276	18.10	5.09		3.47			
			+1.0		18.85	-0.88	+0.12	3.44	.00231	.00919	13
4+38			14.0	255	19.60	5.97		3.41			

(9) The control section moved upstream to the transition section at Sta. 0+50.
The reservoir water surface elevation equaled:

$$\text{Invert El. Station } 0{+}50 + y_c + V_c^2/2g + K_e(V_c^2/2g) = \text{Res. W.S. Elev.}$$

$$1228.43 + 14.0 + 5.97 + 1.19 = 1249.59 \text{ feet msl.}$$

(11) DISCHARGE-SINGLE CONDUIT-RATING CURVE



(10) TABLE II
DISCHARGE COMPUTATIONS FOR PRESSURE FLOW

Reservoir Elev. feet msl. Col. 1	H feet Col. 2	$(H)^{0.5}$ Col. 3	$Q = 2475(H)^{0.5}$ cfs Col. 4
1280	50	7.07	17,500
1300	70	8.37	20.700
1310	80	8.94	22,100
1320	90	9.49	23,500
1360	130	11.40	28,200
1375	145	12.04	29.800

RESERVOIR OUTLET CONDUIT EXAMPLE

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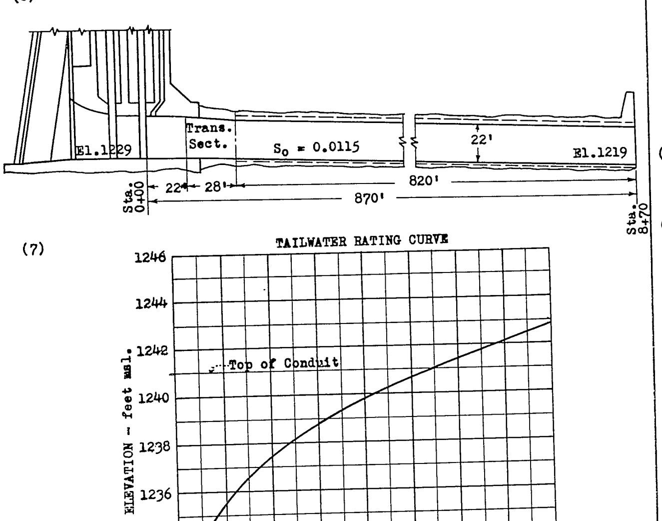
DETERMINATION OF THE DISCHARGE RATING CURVE OF A RESERVOIR OUTLET CONDUIT

Item

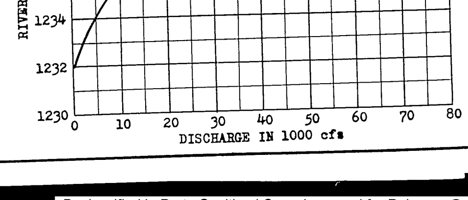
INITIAL DATA

(1) 10 flood control conduits – 22 feet in diameter
(2) 2 intake passages per conduit
(3) Manning's "n" = 0.013
(4) Bottom slope = 0.0115
(5) Total discharge for open channel flow = 50,000 cfs.
Assume equal flow distribution, Q = 5,000 cfs per conduit.
y_c = 14.0 feet for 5,000 cfs in a 22 foot diameter conduit.

(6) PROFILE



(7) TAILWATER RATING CURVE

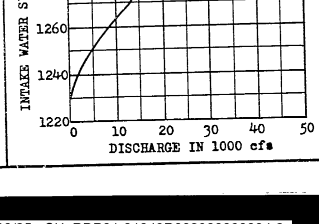


(8) TABLE I
COMPUTATION OF WATER SURFACE PROFILE

Station	Water Surf. Elev. ft. msl	Bottom Elev. ft. msl.	Depth feet	Area feet²	Vel-ocity ft/sec	$\frac{V^2}{2g}$ feet	$\triangle H_o$ feet	$R^{2/3}$	Slope S	$S_o - S$	$\triangle L$ feet
Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11	Col. 12
8+70	1240.6	1219.0	21.6	378	13.23	2.72		3.27			
			+0.6		13.315	-0.07	+0.53	3.325	.001232	.010268	52
8+18			21.0	373	13.40	2.79		3.38			
			+2.0		13.86	-0.40	+1.60	3.46	.001230	.01027	156
6+62			19.0	349	14.32	3.19		3.54			
			+2.0		15.10	-0.73	+1.27	3.545	.001391	.010109	125
5+37			17.0	315	15.88	3.92		3.55			
			+2.0		16.99	-1.17	+0.83	3.51	.001797	.009703	86
4+51			15.0	276	18.10	5.09		3.47			
			+1.0		18.85	-0.88	+0.12	3.44	.00231	.00919	13
4+38			14.0	255	19.60	5.97		3.41			

(9) The control section moved upstream to the transition section at Sta. 0+50.
The reservoir water surface elevation equaled:
Invert El. Station 0+50 + y_c + $V_c^2/2g$ + $K_e(V_c^2/2g)$ = Res. W.S. Elev.
1228.43 + 14.0 + 5.97 + 1.19 = 1249.59 feet msl.

(11) DISCHARGE-SINGLE CONDUIT-RATING CURVE



(10) TABLE II
DISCHARGE COMPUTATIONS FOR PRESSURE FLOW

Reservoir Elev. feet msl.	H feet	$(H)^{0.5}$	$Q = 2475(H)^{0.5}$ cfs
Col. 1	Col. 2	Col. 3	Col. 4
1280	50	7.07	17,500
1300	70	8.37	20.700
1310	80	8.94	22,100
1320	90	9.49	23,500
1360	130	11.40	28,200
1375	145	12.04	29,800

RESERVOIR OUTLET CONDUIT EXAMPLE

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CHAPTER IX
HEADWATER CONTROL

SECTION A: BASIC CONSIDERATIONS

170. Definition. Headwater control pertains to the regulation of the reservoir water surface by means of spillways or outlet conduits. Spillways are normally designed to discharge large rates of flow to protect the dam against the "Spillway Design Flood". Outlet conduits or sluices are normally designed to pass relatively small volumes of flow to regulate the reservoir water surface, or to maintain closely regulated outflows downstream from the dam. The headwater control of a reservoir is accomplished by means of spillway crest gates acting under relatively low heads, and high pressure outlet gates and valves normally operating under high heads.

171. Spillway Crest Gates. Spillway crest gates include two general types of structures:

(1) Gates in which the damming surface is raised to permit discharge between the lower gate lip and the fixed crest; and

(2) Gates in which the damming surface is raised to increase the effective crest level of the spillway, thus regulating the volume of flow over the top surface or edge. Vertical lift gates and tainter gates are the two types commonly identified in the first classification, while flash boards and drum gates are identified as commonly used controls in the second classification.

172. High Pressure Outlet Gates. a. High pressure outlet gates are used to regulate the release of water for irrigation, flood control, domestic and power demands for dams with heads in excess of 75 feet. Gates used for the release of water under lower heads were considered in the paragraphs on spillway crest gates and navigation dams. Figures 8-15 on Plate 901 depict examples of the more common types of vertical lift gates, as follows:

- (1) Slide Gates
- (2) Tractor Type Gates
  - (a) Caterpillar
  - (b) Roller bearing
- (3) Coaster Gate
- (4) Cylinder Gates
- (5) Ring Follower Type Gates
  - (a) Ring follower
  - (b) Paradox
  - (c) Ring seal

b. Gate Chambers. The gate chamber may be defined as the passageway through which the water flows when the gate is open and in which the gate leaf rests when the gate is closed. The cross-sectional shape of the gate chamber in the conduit is normally the same shape as the gate. In some foreign dams the gate is smaller than the conduit and is located in a gradually tapered transition section to reduce the turbulence loss and to prevent separation of the flow.

c. Standard High Pressure Gates. A standard high pressure gate consists of two rectangular heavily ribbed castings, known as the gate frames, which when bolted together form a slot to guide and support the gate leaf. The gate leaf slides vertically along this slot in its opening and closing movements. The gate leaf slides against bronze seal bars on the downstream side of the gate frame. It is from this action that the term "slide gate" is derived. The slot recesses on the gate frames are continued upward into a bonnet casting. The gate leaf, when raised to its open position, is received in the bonnet thereby leaving the water passage open to discharge the flow of water. The gate frames and bonnets are strongly ribbed, both vertically and horizontally to distribute the heavy reactions set up by the hydraulic hoists in opening and closing the gate leaf.

d. Characteristics of High Pressure gates. High pressure gates may be classified by (1) the shape of the water passage through the gate, and (2) the character of service required of them.

(1) Shape. Gates designed for conduits that release large quantities of water are normally rectangular in shape. The hydraulically-operated rectangular slide gate has been used preponderantly for regulating gate installations in sluice conduits of concrete and earth dams. Gates installed in closely regulated conduits of smaller size are normally circular in shape. Irrigation projects in which close regulation of the flow is important, would normally require valve controlled outlet conduits of circular shape, and therefore have circular shaped guard gates to protect the regulation valves. The ring follower gate is an example of a circular gate which has been designed for irrigation projects.

(2) Types of Service. Each type of high pressure outlet gate is normally designed to operate either in the fully opened or completely closed position. The location of the gate with respect to the conduit is dependent on the type of service the gate is expected to perform. Gates may be classified as regulating gates, and guard or emergency gates.

(a) The gate chamber of a regulating or service gate is normally constructed inside the dam proper of a concrete dam, and in the intake structure, or central control tower, for earth dams. Slide gates are normally used as regulating gates in the small conduits of concrete dams, and tractor type gates are used in the relatively large conduits of earth dams.

(b) Guard gates are classified as the type of gate used in protecting a regulating gate or valve from damage during an emergency. They are usually constructed immediately upstream from a regulating gate or valve in the outlet conduit of concrete dams.

Guard gates are usually slide gates or ring follower gates if the conduit is relatively small, and tractor type of coaster gates if the conduit is large. The tractor type gate was designed for high head dams where large quantities of water were to be passed through penstocks and the gates were to be installed on the upstream face of the dam. Gates of this type have been designed for rectangular inlets 16.5 feet by 28.5 feet and for heads over 180 feet. Cylinder gates are a type of slide gate that is curved in plan to fit a circular intake tower of a high head dam.

173. High Pressure Outlet Valves. High pressure outlet valves may be divided into two classes, (1) regulating valves, and (2) guard valves. Regulating valves, as their name implies, are designed to give close regulation of the outflow from high pressure outlet conduits. They are principally used on irrigation projects where the release of relatively small quantities of water under close control is of primary importance Guard valves are used to protect regulating valves or turbines in case of a sudden breakdown, or for dewatering the outlet conduit or penstock for maintenance and repair service.

a. Regulating Valves. The common types of regulating valves are shown as Figures 16 to 19 on Plate 901 and are listed as follows:

(1) Needle valves
  (a) Balanced needle
  (b) Internal differential needle
  (c) Interior differential needle
(2) Tube valves
(3) Howell & Bunger valves
(4) Hollow-jet valves

A needle type valve opens and closes by the horizontal movement of a needle, which is closed when the needle is advanced to its extreme downstream position. The water flows in an annular passageway, first diverging then converging past the needle. The different types of needle valves are basically the same, differing only in the mechanics of the needle movement. A tube valve is a needle type valve with the downstream portion of the needle eliminated. The hollow jet valve is a further modification of the needle valve in which the flow is not contracted after it passes the needle, and the needle moves upstream to close against the outer casing of the valve. The Howell and Bunger valve is similar to the hollow jet valve. The cone of the Howell and Bunger valve points upstream and is stationary on the downstream end of the valve. A sleeve on the outer casing of the valve moves downstream to close the valve. The outflow from a Howell and Bunger valve or a hollow jet valve is a hollow-centered diverging jet which creates considerable spray downstream of the outlet portal.

b. Butterfly Valves. A butterfly valve, as shown in Fig. 20, of Plate 901, is a type of guard valve which has been extensively used in power penstocks and to some extent in free discharge outlet conduits.

A butterfly valve consists of a gate leaf which is normally the same shape and size as the conduit cross section. The gate leaf disc turns on a central axis which is normal to the flow of water. When in the full open position, the leaf disc is turned 90° with its face parallel to the flow. The shape of the immersed butterfly leaf should be streamlined to reduce the head loss when in the open position.

SECTION B: SPILLWAY CREST GATES

174. Factors Involved. This section will present the method of computing a discharge rating curve for spillway crest gates. The gates are assumed to be on the crests of high dams and therefore not affected by the tailwater elevation. The discharge from submerged sluice and tainter gates were discussed in Pars. 150 and 154, for navigation and low head dams.

175. Discharge Capacity. a. The discharge capacity for a partially opened vertical gate or tainter gate is normally computed by the basic orifice equation:

$$Q = C_q bL\ (2gh)^{0.5} \qquad (9\text{-}1)$$

where

- Q = discharge in cfs
- h = the effective depth of water upstream from the gate in feet
- L = gate width in feet
- b = the effective gate opening in feet
- C_q = a variable coefficient of discharge

b. The discharge capacity for a drum gate is computed by the basic weir equation:

$$Q = C_q LH^{1.5} \qquad (9\text{-}2)$$

where

- Q = discharge in cfs
- L = the effective crest width in feet
- H = the total head on the gate crest in feet
- C_q = a variable coefficient of discharge

176. Vertical Lift Gates. Spillway crest gates of the vertical lift type are normally designed as slide gates or roller bearing tractor gates. In computing the discharge under a partially opened gate, the gate lip is assumed to have the same discharge characteristics as a sharp edged sluice.

a. Discharge Coefficient. The coefficient of discharge /1/ of a vertical lift gate is a function of the ratio of the head on the gate to the gate opening, and the coefficient of contraction. The coefficient of discharge of a vertical gate was determined from experiments as a function of the ratio of the head on the gate to the effective gate opening, and is shown on Plate 902. Curves showing the relationship of the discharge coefficient for inclined gates with the angle of inclination varying from 15° to 90° are also shown on Plate 902.

b. Discharge Rating Curve. The discharge rating curve for partially opened vertical lift gates on a spillway crest would be computed as follows:

(1) Construct a rating curve for all gates fully open as described in Par. 103.

(2) Draw the spillway crest and gate profiles to scale.

(3) Assume a gate opening (measured vertically from the bottom edge of the gate to the gate seat).

(4) Determine the effective gate opening (measured from the bottom edge of the gate to the nearest point of spillway crest) as described in Par. 146.

(5) Assume a reservoir water surface elevation.

(6) The effective head is computed as the depth of water from the water surface to the crest point which was nearest the gate lip, as determined in (4) above.

(7) Determine the coefficient of discharge from Plate 902. The discharge coefficient would be determined for an inclined gate with θ being measured as the angle between the tangent to the spillway crest at the crest point, and a vertical plane. The ratio of h/b would be based on the effective gate opening and depth as determined in (4) and (6) above.

(8) Compute the discharge for the effective gate opening and the assumed reservoir water surface elevation by equation 9-1.

(9) Assume other pool elevations and compute the discharge for the given gate opening.

(10) Superimpose the rating curve for the effective gate opening on the rating curve of (1) above. The curve should be labeled according to the nominal gate opening and not the effective opening.

(11) Other gate openings should be assumed and the above procedure repeated.

177. Tainter Gates. The discharge characteristics of a partially opened tainter gate is essentially the same as for a vertical lift gate. The primary difference in the discharge characteristics of the two type gates is due to the curved damming surface of the tainter gate as compared to the flat surface of the sluice gate. The curved surface of the tainter gate causes less contraction of the upper nappe than a vertical sluice gate and therefore has a greater discharge capacity for the same head. The effective gate lip angle θ (described in Par. 148) is a function of the gate opening, the tainter gate arm length, the height at which the trunnion pin is set above the gate seat, and the shape of the spillway crest.

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a. Discharge Coefficient. The discharge coefficient /1/ has been determined by experiment for tainter gates in the same manner as for vertical and inclined sluice gates as described in Par. 176. Curves showing the relationship between the angle of the tainter gate lip with the tangent to the spillway at the nearest crest point, are given on Plate 902.

b. Discharge Rating Curve. The method of computing a discharge rating curve for a tainter gated spillway would be generally the same as described in Par. 176. The discharge rating curve for a partially opened tainter gate on a spillway crest would be computed as follows:

(1) Follow the same procedure as for a vertical lift gate from steps (1) through (6) as described in Par. 176.

(2) Determine the coefficient of discharge from Plate 902. The gate lip angle (θ) would be computed as follows:

$$\text{Cos}\ (\theta) = \frac{a - b}{r} \qquad (9\text{-}3)$$

where

θ = the gate lip angle as defined above
a = the trunnion pin height above the gate seat
b = the gate opening above the gate seat
r = the radius of the tainter gate

The discharge coefficient is determined from the effective gate lip angle θ which is the angle between the tangent to the nearest spillway crest point and the tangent to the tainter gate lip, as described i Par. 148. The ratio of h/b would be based on the effective gate opening and head, as described in paragraphs 146 and 147.

(3) The remainder of the procedure would be the same as that for a vertical lift gate as described in Par. 176, steps (8) through (11).

The discharge rating curves of gated spillways have been shown by model tests to vary in discharge at certain pool levels depending on whether the pool was rising or falling. In a falling pool, the spillway nappe clings to the gate lip resulting in orifice type flow. In a rising pool the spillway nappe would not impinge on the gate lip, thus resulting in weir type flow with increased discharge. A typical discharge rating curve for a tainter gated spillway is shown on Plate 903.

178. Drum Gates. A drum gate is a type of crest control that has the cross section of a circular sector, and is housed in the crest of the spillway. The gate is hinged at the center of curvature, either upstream or downstream, in such a manner that the entire gate may be raised above the masonry crest or lowered so the upper surface becomes coincident with the crest line. A typical cross section of a drum gate is shown on Plate 904. When raised,

the drum gate simulates a sharp crested weir with a curved upstream face. When lowered, the gate offers no appreciable obstruction to flow, and the discharge is computed as for a standard ogee crest.

a. Discharge Capacity. The discharge capacity over a drum gate is computed by means of the basic weir equation:

$$Q = C_q L H^{1.5} \qquad (9\text{-}4)$$

as described in Par. 175.

b. Coefficient of Discharge. (1) The discharge coefficient of a drum gate is a function of three variables: (1) the angle between a line drawn tangent to the downstream lip of the drum gate and a horizontal plane, (2) the radius of the gate r, or the equivalent radius of curvature if the gate crest is not circular, and (3) the total head on the gate.

(2) A series of model tests /2/ were made on eleven drum gates to determine the variation in the coefficient of discharge for partial gate openings. Plate 904 shows the discharge coefficient as a function of θ, the angle between the tangent to the gate lip and the horizontal; and also as a function of the ratio of the total head to the radius of the gate. The depth of approach was not included as a variable. When the approach depth, measured below the high point of the gate, is equal to or greater than twice the head on the gate, a further increase in approach depth produces very little increase in the coefficient of discharge. Most drum gates are elevated sufficiently so that the approach depth is not effective. Also the effect of submergence due to tailwater was assumed to be negligible.

(3) The downstream lip of a drum gate does not control the flow for negative values of the angle (θ). The control point shifts upstream to the vicinity of the high point of the gate for each gate setting, and flow conditions gradually approach those of a free crest as the gate is lowered. The drum gate positions are shown on Plate 904 for positive and negative gate lip angles, and with the control point indicated by the high point of the gate. The gate position has little effect on the discharge in the region of $\theta = -15^{\circ}$ and when the gate is completely closed.

c. Discharge Rating Curves. The discharge rating curve for a drum gate would be computed as follows:

(1) Construct a rating curve for all gates fully closed by the methods described in Par. 103 for an ogee crested spillway.

(2) From a profile of the drum gate, determine the radius of the drum gate, the horizontal distance from pin to gate lip, and the elevations of the gate pin, crest, and downstream lip.

(3) Compute the chord length and the initial angle of depression α by elementary geometry and trigonometry.

(4) Compute the initial angle θ for the gate closed. Cos. θ = the difference in elevation of the gate lip and the center of curvature of the gate, divided by the radius of the gate.

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(5) Assume various elevations of the gate lip and compute the angle β. $\sin \beta$ = difference in elevation between the gate lip and pin, divided by the chord length.

(6) The difference between the absolute values of the sum of α and β, and the initial angle θ when the gate is in the closed position, is the gate angle for any gate opening.

(7) Compute the elevation of the high point of the gate, equal to the elevation of gate lip + (r - r Cos θ). Where r = radius of the gate and θ = gate angle for the assumed gate openings.

(8) Assume a gate opening and a reservoir elevation and determine the discharge coefficient from Plate 904. The head is measured from the water surface to the high point on the gate.

(9) Compute the discharge by equation 9-4.

(10) Repeat steps (8) and (9) for different reservoir elevations and plot the rating curve for the assumed gate opening.

(11) Assume different gate openings and repeat steps (8), (9), and (10). This procedure would give a complete rating curve for a drum gate.

179. Examples. The computation of the discharge rating curve for a partially opened tainter gate on a high overfall spillway is shown on Plate 905. The computation of the discharge rating curve for the Black Canyon Dam spillway with a drum gate partially opened is shown on Plate 906.

SECTION C: HIGH PRESSURE OUTLET GATES

180. Factors Involved. High pressure outlet gates are normally designed to operate either fully open or completely closed as has been previously stated. With the gates completely open, the discharge rating curve would be computed by the methods given in Chapter VIII, Reservoir Outlet Conduits. For military hydrology purposes in artificial flooding operations, it may be advantageous to regulate the flow by partial gate openings.

181. Discharge Capacity. The discharge capacity under a partially opened sluice gate would be computed by the basic orifice equation:

$$Q = C_q bL \quad (2gH)^{0.5} \qquad (9\text{-}5)$$

as described in Par. 175, with the head "H" described below.

182. Head. The head on a partially opened sluice gate, that is located at the entrance of an outlet conduit, is measured from the reservoir water surface to the top of the vena contracta of the jet. If the gate was located downstream from the entrance of the conduit, the head would be measured from the energy gradient just upstream from the gate to the vena contracta of the jet.

183. Discharge Coefficient. The coefficient of discharge of a slide gate or tractor gate is sensitive to the shape of the gate lip. The leakage around a large gate for partial openings may also materially affect the discharge coefficient. The discharge coefficient of a partially opened gate is a function of the percent of gate opening and the contraction of the jet. Data /3/ were taken from tests on model and prototype structures having various gate clearances and lip shapes. An average design curve has been plotted for the coefficient of discharge as a function of the percent of gate opening as shown on Plate 907.

184. Discharge Rating Curve. The discharge rating curve for a partially opened high pressure gate would be a family of curves plotted as reservoir elevation against discharge with each curve representing a given gate opening.

a. Gate at Entrance of Conduit. The discharge rating curve for a partially opened gate located near the entrance of the outlet conduit would be computed as follows:

(1) Assume a gate opening and determine its percent of full gate opening.

(2) Determine the coefficient of discharge from Plate 907 for the ~~percent of gate opening~~ of step (1) above.

(3) Assume a reservoir water surface elevation.

(4) Determine the head on the sluice as the difference in elevation from the assumed reservoir water surface elevation to the top of vena contracta of the issuing jet. The top of the vena contracta is 0.6 of the gate opening.

(5) Compute discharge for the assumed reservoir elevation and gate opening by equation 9-5.

(6) Repeat the above procedure for various reservoir elevations and plot the discharge rating curve for the given gate opening.

(7) Repeat the above procedure for different gate openings and plot the family of curves.

b. Gate Located Downstream From Entrance. The discharge rating curve for a partially opened gate located considerably downstream from the conduit entrance would be computed basically the same as described above. The difference is that the head would be computed from the energy gradient instead of the reservoir elevation. The elevation of the energy gradient at the gate would be the difference between ~~the reservoir water surface elevation~~ and the head loss due to the entrance losses, bend and gate losses, and ~~friction in~~ the conduit. The discharge rating curve of a partially opened gate located near the outlet of a conduit would be computed as follows:

(1) Assume a gate opening and determine the discharge coefficient as in steps (1) and (2) in subparagraph 184a above.

(2) Assume an elevation of the energy gradient immediately above the partially opened gate.

(3) Determine the head on the sluice gate as the difference in elevation from the assumed energy gradient to the vena contracta of the jet and compute the discharge as in steps (4) and (5) of subparagraph 184a above.

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(4) Determine the velocity head coefficients for the conduit, entrance losses, transition losses, friction loss, and any bend or gate recess losses above the partially opened gate. The above coefficients are determined by methods described in Chapter VIII.

(5) Compute the velocity head in the conduit for the discharge computed in step (3) above.

(6) Determine the head loss for the discharge as the product of the sum of the velocity head coefficients of step (4) and the velocity head of step (5).

(7) The total loss of head added to the assumed energy gradient would give the reservoir water surface elevation necessary to deliver the discharge of step (3).

(8) Plot the discharge against the required reservoir water surface elevation.

(9) Repeat the above procedure for different energy gradient elevations and plot the rating curve for the given gate opening.

(10) Repeat the above procedure for different gate openings and plot the family of curves.

185. Example. The computation of the discharge rating curve of an outlet conduit with a regulating gate near the outlet is shown on Plate 908.

SECTION D: HIGH PRESSURE OUTLET VALVES

186. Factors Involved. High pressure outlet valves are normally designed to give close regulation for relatively small rates of flow. The butterfly valve is the exception to the above and is normally designed as a guard valve for turbines and regulating valves.

187. Discharge Capacity. a. The method of computation of the discharge capacity of a high pressure regulating valve is dependent on where the valve is located with respect to the downstream portal.

b. If the valve is located at the downstream end of the conduit and discharges into the atmosphere, the rate of flow is computed by the basic orifice type equation:

$$Q = C_q A \quad (2gH)^{0.5} \qquad (9\text{-}6)$$

where

- Q = the discharge in cfs
- A = the cross-sectional area of the nozzle outlet in sq. ft.
- H = the effective head on the center line of the valve inlet flange in ft.
- C_q = the coefficient of discharge and is dependent on the type of valve considered.

c. If the valve is located upstream of the outlet portal and discharges into the conduit, the discharge is computed as a head loss for a valve in a pipe. The discharge is computed by the basic pipe flow equation, and the valve head loss is a function of the velocity head and is computed as follows:

$$h_v = K_v \frac{V^2}{2g} \qquad (9\text{-}7)$$

where

- h_v = the head loss due to the valve in feet
- K_v = the velocity head coefficient and is dependent on the type of valve
- V = the velocity of flow at the valve outlet in ft/sec

188. Discharge and Velocity Head Coefficients. a. Discharge Coefficients. The discharge coefficients of high pressure outlet valves are normally given for the full-open position and with free discharge into the atmosphere. The values of the coefficient of discharge /4/ to be used in equation 9-6 for free flow conditions are as follows:

TYPE OF VALVE		C_q
(1) Sharp-edged nozzle orifice (Balanced needle)	=	0.65
(2) Rounded-edged nozzle orifice (Internal differential and Interior differential)	=	0.78
(3) Short-bodied tube valve	=	0.60
(4) Long-bodied tube valve	=	0.76
(5) Howell and Bunger valve	=	0.90
(6) Hollow jet valve	=	0.70

For military hydrology purposes the above coefficients of discharge would be assumed to vary in direct proportion to the valve opening for each partially opened valve. Experiments /3/ on Howell and Bunger valves have shown that the discharge coefficient curve is nearly a straight line and varies approximately in direct proportion to the sleeve opening.

b. Velocity Head Coefficients. The value of the velocity head coefficients to be used in equation 9-7 for various types of valves are listed below:

(1) Needle Valves. /5/

$$K_v = 0.183 \ (l/d)^{1/3} \qquad (9\text{-}8)$$

where

- K_v = the velocity head coefficient
- d = the diameter of the valve outlet in feet

(2) Butterfly Valves. /5/

$$K_v = t/d \qquad (9-9)$$

where

t = the thickness of the disk of the valve in ft.
d = the diameter of the valve in ft.

(3) Gate Valves. /6/

Valves	K_v
Fully open	0.19
1/4 closed	1.15
1/2 closed	5.6
3/4 closed	24.0

(4) Globe Valve. /7/

Fully open	10.0

(5) Angle Valve.

Fully open	5.0

(6) Swing Check Valve.

Fully open	2.5

189. References.

/1/ Gentilini, Bruco. "Ecoulement sous les vannes de fond inclinees ou a secteur—resultats tecjniques et experimentaux". La Houille Blanche. Vol. 2, 1947, pp 145-149. ("Flow Under Inclined or Radial Sluice Gates". Corps of Engineers, Research Center, Waterways Experiment Station, Vicksburg, Mississippi. Translation No. 51-9).

/2/ Bradley, Joseph N. "Rating Curves for Flow Over Drum Gates". Proceedings A.S.C.E. Vol. 79, Separate No. 169, Feb. 1953.

/3/ "Hydraulic Design Criteria". Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi.

/4/ Davis, Calvin Victor. Handbook of Applied Hydraulics. McGraw-Hill Book Company, 1942. pp 393-456.

/5/ Creager, William P., and Joel D. Justin. Hydroelectric Handbook. John Wiley & Sons, 1950. p 106.

/6/ "Engineering Experimental Station Bulletin", University of Wisconsin, Vol. 9, No. 1, 1922.

/7/ "Flow of Fluids Through Valves, Fittings, and Pipes". Engineering and Research Division, Crane Co., Chicago. Tech. Paper No. 409, May 1942.

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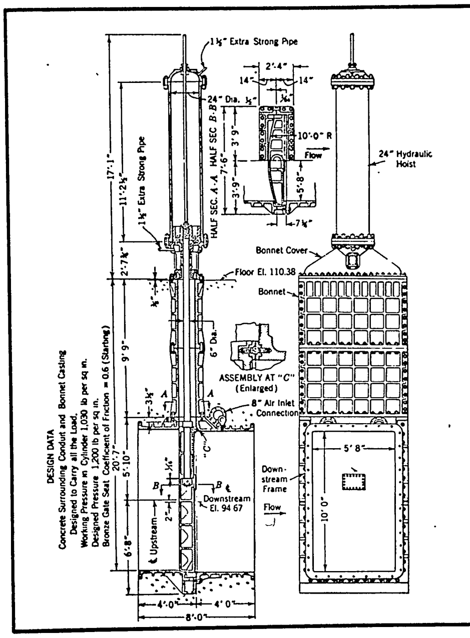

Figure 8

TYPICAL SLIDE GATE

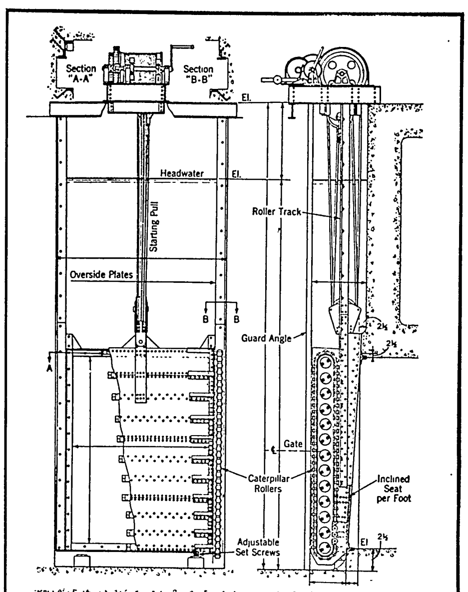

Figure 9

TRACTOR TYPE CATERPILLAR GATE

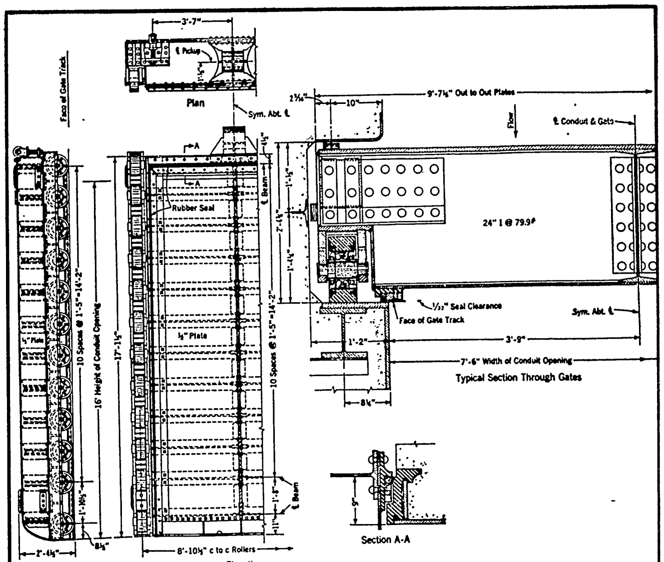

Figure 10

TRACTOR TYPE ROLLER BEARING GATE

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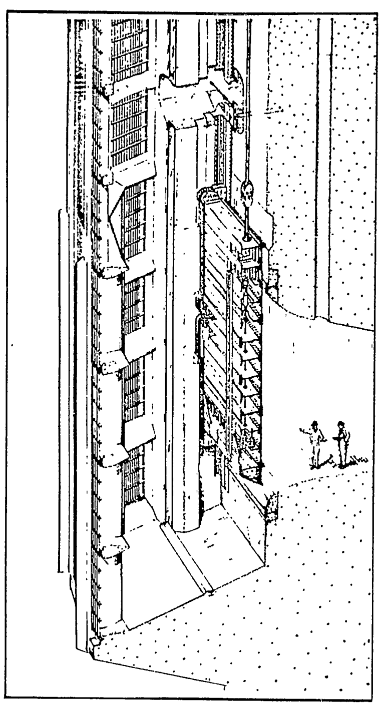

Figure 11

15 by 29.65 ft. STEM-OPERATED COASTER GATE

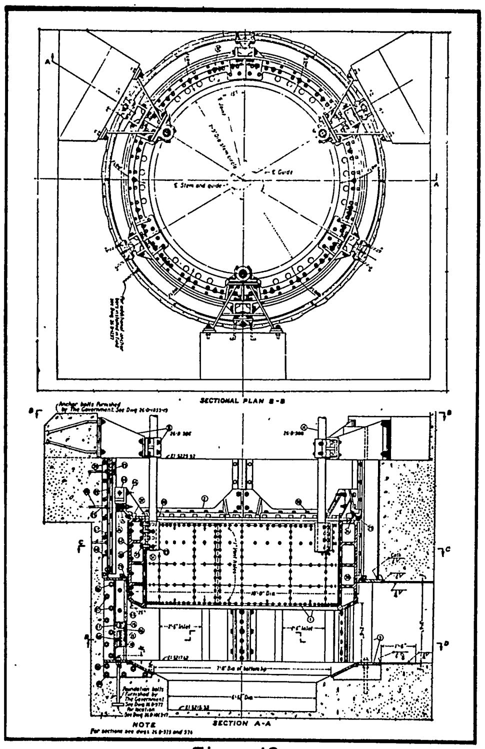

Figure 12

TEN FOOT CYLINDER GATE ASSEMBLY

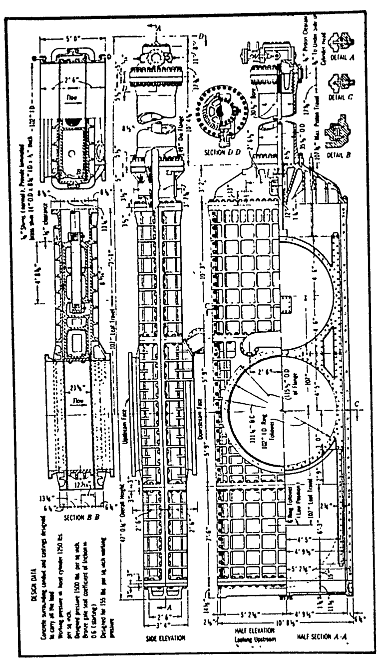

Figure 13

RING FOLLOWER EMERGENCY GATE

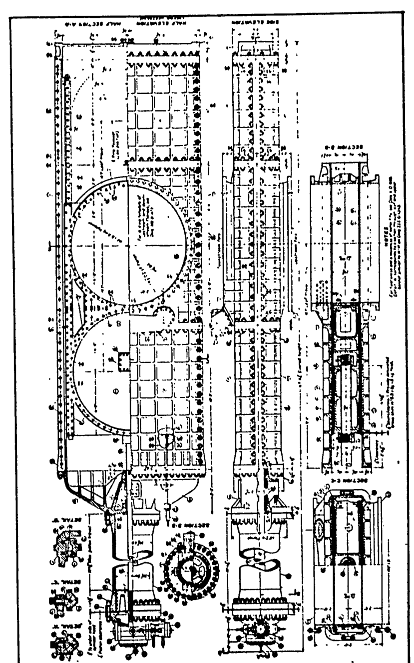

Figure 14

102 INCH PARADOX MOTOR OPERATED SERVICE GATE

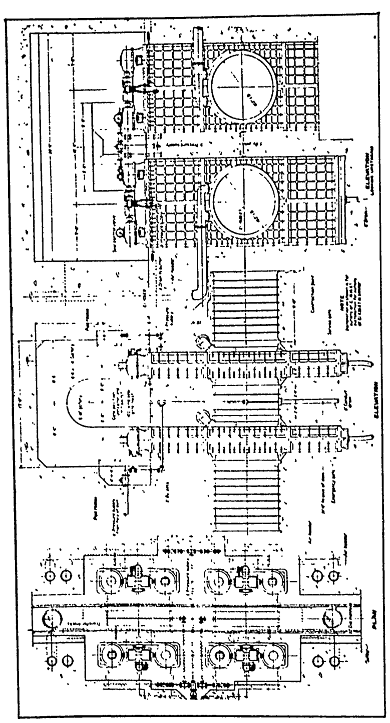

Figure 15

102 INCH RING SEAL EMERGENCY & SERVICE GATES

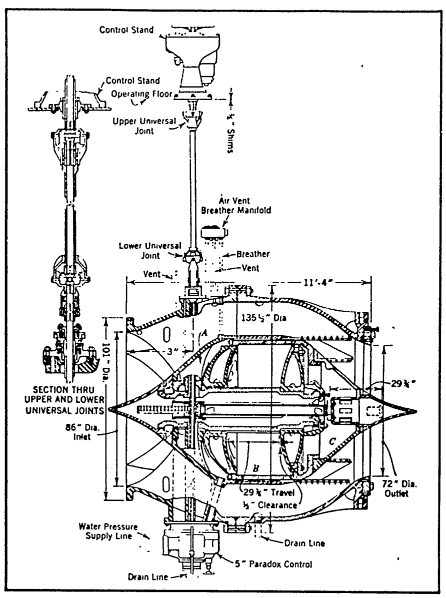

Figure 16

NEEDLE VALVE
(INTERNAL DIFFERENTIAL)

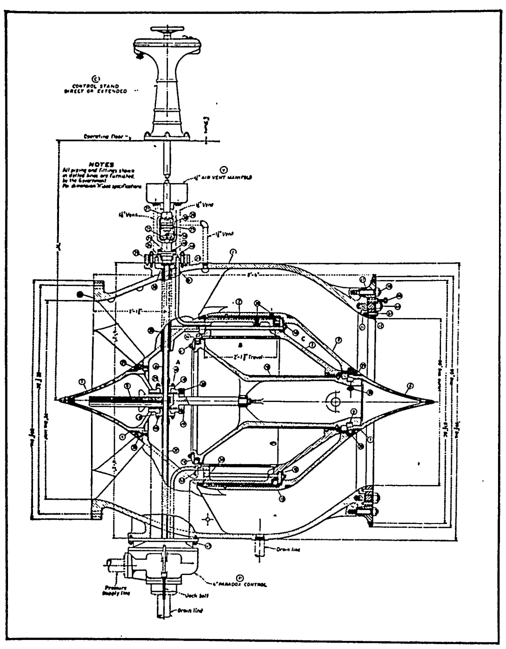

Figure 17

NEEDLE VALVE
(INTERIOR DIFFERENTIAL)

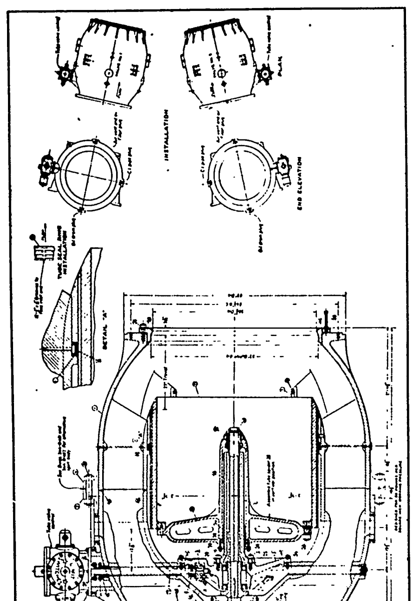

Figure 18.

TYPICAL INLET TUBE VALVE, WITH DIRECT MOUNTED CONTROL

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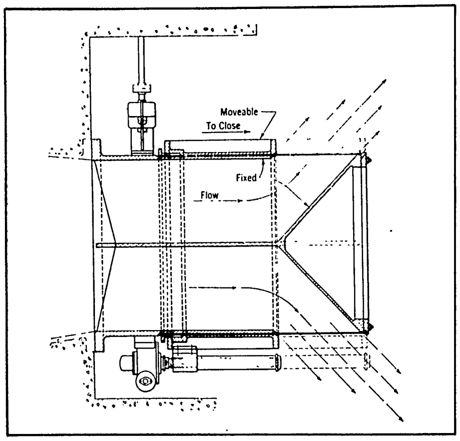


Figure 19

HOWELL-BUNGER VALVE

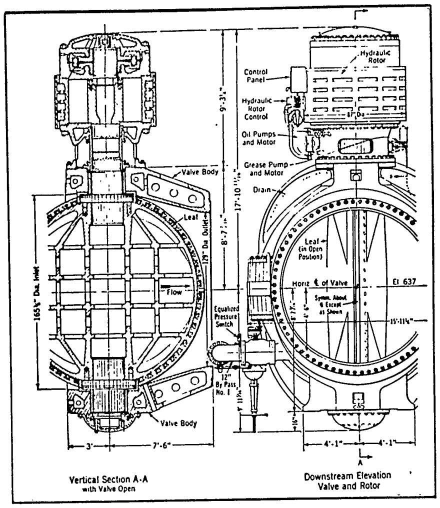
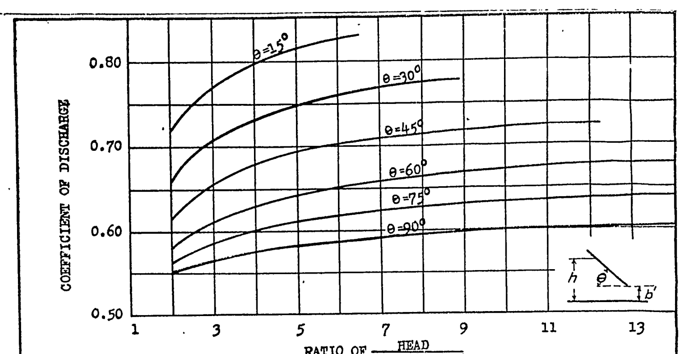
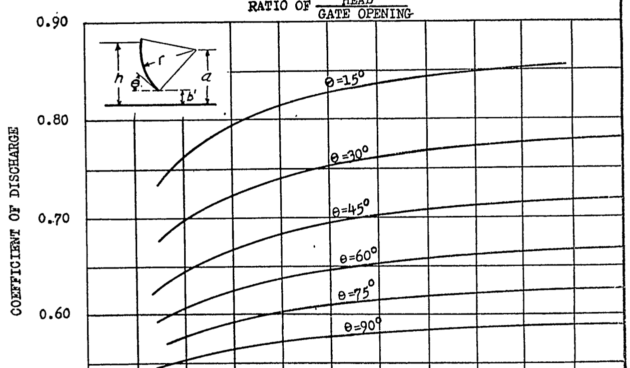


Figure 20

BUTTERFLY VALVE & ROTOR





INCLINED & RADIAL GATE DISCHARGE COEFFICIENT

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WASHINGTON DISTRICT CORPS OF ENGINEERS
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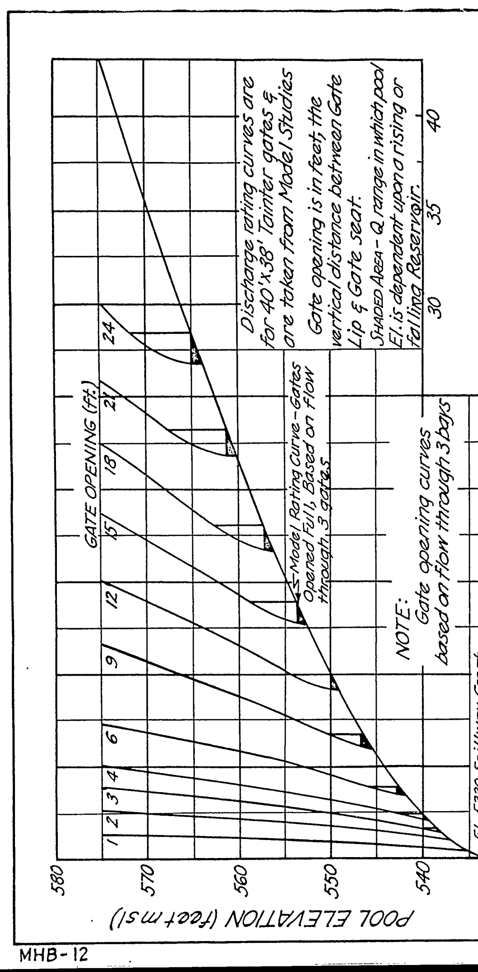
Drawn by ____________

MHB-12

PLATE 902



GATED SPILLWAY DISCHARGE RATING CURVES

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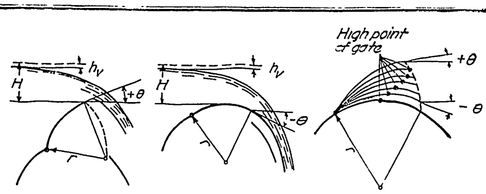
MHB-12

PLATE 903



(a) POSITIVE ANGLE θ (b) NEGATIVE ANGLE θ (c) CONTROL POINT

Fig. 1, DRUM GATE POSITIONS

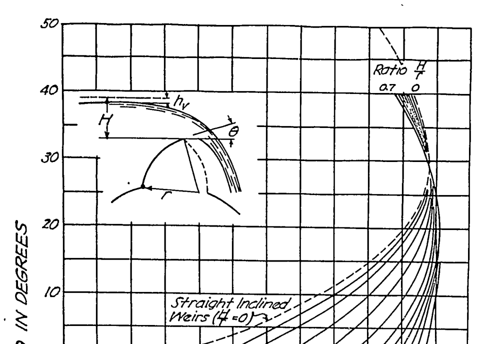


Fig. 2, DRUM GATE DISCHARGE COEFFICIENTS

DRUM GATE X-SECTION & DISCHARGE COEFFICIENTS

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Drawn by ____________

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PLATE 904

DETERMINATION OF THE DISCHARGE RATING CURVE FOR A PARTIALLY OPEN TAINTER GATE ON A HIGH OVERFALL SPILLWAY

EXPLANATION OF COMPUTATIONS

<u>Item</u>

INITIAL DATA

(1)-(2) The dimensions and elevations of the spillway crest and tainter gate are shown on Plate 905 D.

(3) The discharge rating curve for the tainter gate completely open was determined by the methods given in Chapter VI and is shown on Plate 905 D.

EFFECTIVE GATE OPENING AND PROFILE POINT ELEVATION

(4) The spillway profile and tainter gate was drawn to scale as shown on Plate 905 D. The nominal gate openings were assumed as shown in Col. 1, Table I. For each vertical gate opening listed in Col. 1, the minimum distance from the gate lip to the spillway profile was determined and tabulated in Col. 2. The distance from the spillway crest to the point on the spillway profile nearest the gate lip, as determined in Col. 2, was scaled and tabulated in Col. 3. The elevation of the point on the spillway profile nearest the gate lip was determined for each gate opening and listed in Col. 4.

EFFECTIVE GATE OPENING AND GATE ANGLE

(5) The assumed nominal gate openings and the effective gate openings determined in Table I were entered in Cols. 1 and 2, respectively of Table II.

The angle between the tangent to the gate lip and the horizontal was computed for each gate opening as follows:

1. <u>Gate lip angle for 2.0 ft. nominal gate opening</u>

$$\text{Cos } \theta = \frac{\text{difference in elev. of trunnion pin and gate lip}}{\text{radius of curvature of tainter gate}}$$

$$= \frac{1551.0 - 1542.23}{27.0} = 0.325$$

$$\theta = 71.0$$

The gate lip angle for each gate opening was computed in the same manner as for the 2.0 foot nominal opening and tabulated in Col. 3 and Col. 4.

The angle between the tangent to the spillway at the point on the profile nearest the gate lip and the horizontal was computed for each gate opening as follows:

1. Angle of spillway tangent for 2.0 ft. gate opening

The slope of the tangent to the spillway at any point was computed from the first derivative of the equation of the spillway profile.

$y = x^2/78$ (Spillway Profile Equation)

$\frac{dy}{dx} = x/39$ (First Derivative of Profile Eq.)

$\text{Tan } \theta = \frac{6.75}{39}$

$= 0.173$ (From Col. 3, Table I)

$\theta = 9.8^\circ$

The spillway tangent angle was computed for each gate opening in the same manner as for the 2.0 foot nominal opening and entered in Col. 5 and Col. 6. The angle of the spillway tangent could have been determined graphically by constructing the tangent to the spillway and measuring the angle with a protractor or determining the length of the sides of the right triangle and computing the tangent of the angle.

The effective gate lip angle was computed as the difference between the gate lip angle listed in Col. 4 and the spillway profile angle listed in Col. 6, and entered in Col. 7.

HEAD-DISCHARGE COMPUTATIONS FOR TAINTER GATE PARTIALLY OPEN

(6) The discharge rating curve for a partially opened tainter gate was computed as described in the following steps:

1. The effective heads for the various gate openings were assumed as shown in Col. 1 of Table III.
2. The ratio of the effective heads in Col. 1, Table III, and the effective gate openings of Col. 2, Table II were computed and entered in Col. 2, Table III.
3. The coefficients of the discharge of Col. 3 were determined from Plate 902 as a function of the values of the effective gate lip angle of Col. 7, Table II and the ratios of the head and effective gate opening of Col. 2, Table III.

4. The discharge was computed by equation 9-1 and entered in Col. 4. The coefficient of discharge was taken from Col. 3, Table III; the effective gate opening was taken from Col. 2, Table I; the gate width was taken from the initial data; and the effective head was taken from Col. 1, Table III.
5. The reservoir water surface elevations tabulated in Col. 5 were determined for each discharge as the sum of the effective heads listed in Col. 1, Table III and the elevation of the points on the spillway profile nearest the gate lip listed in Col. 4, Table I.

DISCHARGE RATING CURVE

(7) The discharge rating curve was plotted for each gate opening from the data tabulated in Col. 4 and Col. 5, Table III and shown on Plate 905 D

DETERMINATION OF THE DISCHARGE RATING CURVE FOR A PARTIALLY OPEN TAINTER GATE ON A HIGH OVERFALL SPILLWAY

INITIAL DATA

Item

(1) Spillway profile and tainter gate dimensions as shown

(2) Tainter gate 64 ft. wide x 28 ft. high.

(3) Discharge rating curve for gate completely open as shown on Sheet 2.

(4)

TABLE I

EFFECTIVE GATE OPENING AND PROFILE POINT ELEVATION

Nominal gate Opening b' Col. 1	Effective gate Opening b Col. 2	Distance of Minimum Profile from Axis Col. 3	Elevation of Minimum Profile Point Col. 4
2.0	1.85	6.75	1540.43
4.0	3.72	5.90	1540.55
6.0	5.66	5.20	1540.65
8.0	7.61	4.65	1540.72

(5)

TABLE II

EFFECTIVE GATE OPENING AND GATE ANGLE

Nominal Gate Opening ft. Col. 1	Effective Gate Opening ft. Col. 2	Cos (Gate Lip Angle) $= \frac{a - b}{r}$ Col. 3	Gate Lip Angle degrees Col. 4	Tan (Spillway Angle) = x/39 Col. 5	Spillway Profile Angle degrees Col. 6	Effective Gate Lip Angle degrees Col. 7
2.0	1.85	0.325	71.0	0.173	9.8	61.2
4.0	3.72	0.251	75.5	0.151	8.5	67.0
6.0	5.66	0.177	79.8	0.134	7.6	72.2
8.0	7.61	0.103	84.1	0.119	6.8	77.3

Item

(6)

TABLE III

HEAD DISCHARGE COMPUTATIONS FOR TAINTER GATE PARTIALLY OPEN

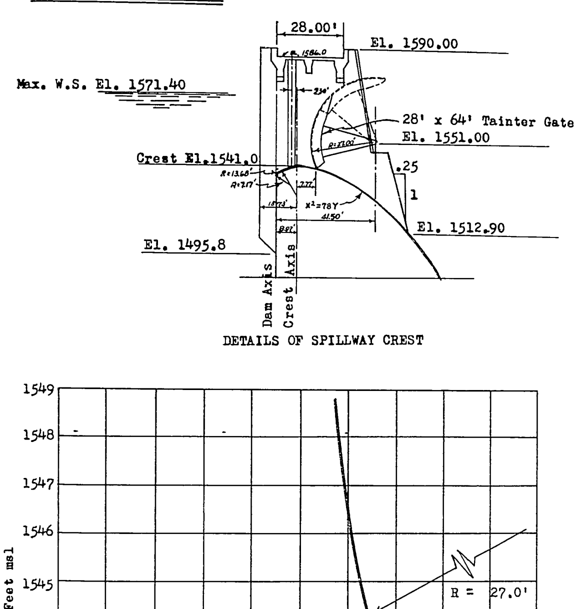
Effective Head ft. Col. 1	Ratio h/b Col. 2	C_q Col. 3	Discharge = $C_q bL \sqrt{2gh}$ cfs Col. 4	Reservoir Water Surface Elevation ft. msl Col. 5
2.0 ft. Nominal Gate Opening				
10	5.4	0.64	1920	1550.43
15	8.1	0.65	2390	1555.43
20	10.8	0.66	2800	1560.43
25	13.5	0.67	3160	1565.43
30	16.2	0.67	3460	1570.43
4.0 ft. Nominal Gate Opening				
10	2.69	0.59	3560	1550.55
15	4.03	0.61	4500	1555.55
20	5.37	0.62	5300	1560.55
25	6.70	0.63	6010	1565.55
30	8.05	0.64	6690	1570.55
6.0 ft. Nominal Gate Opening				
15	2.65	0.58	6550	1555.65
20	3.53	0.59	7700	1560.65
25	4.40	0.60	8760	1565.65
30	5.30	0.61	9730	1570.65
8.0 ft. Nominal Gate Opening				
20	2.62	0.57	9950	1560.72
25	3.28	0.58	11400	1565.72
30	3.94	0.59	12650	1570.72

SPILLWAY CREST GATE EXAMPLE

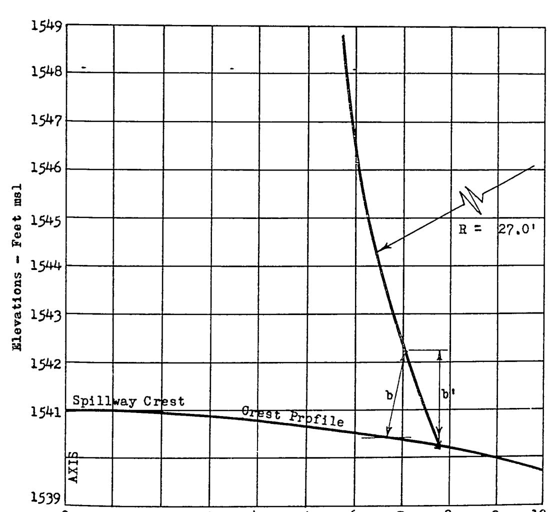
MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ________
Drawn by ____________

PLATE 905C

All elevations ft. msl.

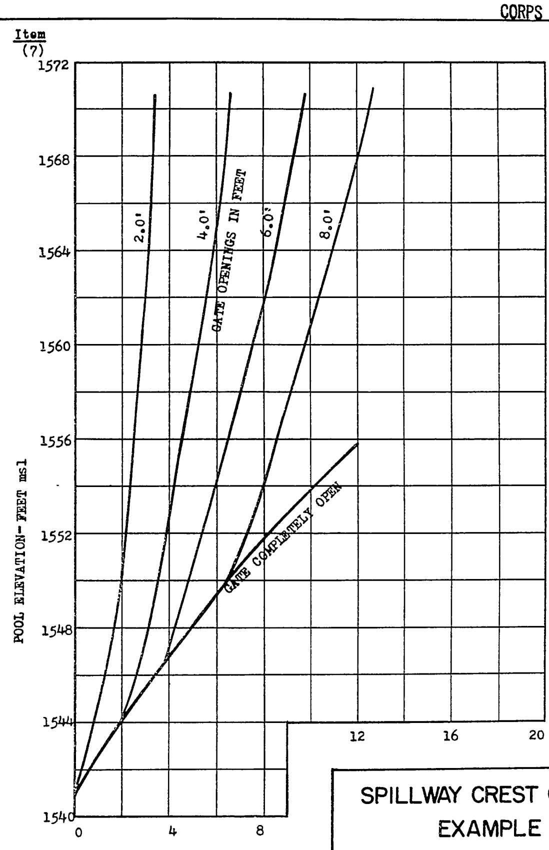


DETAILS OF SPILLWAY CREST



SPILLWAY AND TAINTER GATE PROFILES

Item (7)



DISCHARGE RATING CURVE

SPILLWAY CREST GATE
EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ________
Drawn by ____________

DETERMINATION OF THE DISCHARGE RATING CURVE
FOR BLACK CANYON DAM IN IDAHO

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1) The dimensions and elevations of the Black Canyon spillway crest are shown on Plate 906 E.

(2) The discharge rating curve, for the drum gates completely closed, is shown on Plate 906 E, and was determined by the methods explained in Par. 103.

GATE ANGLE θ AND ELEVATION OF HIGH POINT OF GATE

(3)-(10) The gate lip elevations were assumed as shown in Col. 1, Table I and on Plate 906 E.

The gate angle θ of items (3) and (4) were computed as described in the following steps and entered in Col. 4:

1. Initial angle θ with gate down

$$\cos\theta = \frac{\text{difference in elevation of gate lip and center of curvature}}{\text{radius of curvature}}$$

$$= \frac{2478.2 - 2461.5}{21.0} = 0.795$$

$$\theta = 37.3^{\circ}$$

2. Initial angle α with gate down

$$\tan\alpha = \frac{\text{difference in elev. of pin and gate lip}}{\text{horiz. dist. between pin and gate lip}}$$

$$= \frac{2481.63 - 2478.20}{17.27} = 0.198$$

$$\alpha = 11.2^{\circ}$$

3. Angle β with gate lip at elevation 2485.0 ft. msl.

$$\sin\beta = \frac{\text{difference in elev. of gate lip and pin}}{\text{chord length}}$$

$$= \frac{2485.0 - 2481.63}{17.58} = 0.192$$

$$\beta = 11.0^{\circ}$$

4. Angle Θ with gate lip at elevation 2485.0 ft. msl.

$$\Theta = (\text{Angle } \alpha + \text{Angle } \beta) - \text{Angle } \Theta \text{ (gate in closed position)}$$

$$= (11.2^\circ + 11.0^\circ) - 37.3^\circ$$

$$\Theta = -15.0^\circ$$

Items (5) and (10) were computed in the same manner as item (4) above.

The elevation of the high point of the gate for item (4) was computed as described in the following steps and entered in Col. 5:

1. Elevation of high point of gate.

$$\text{El. of high point} = \text{El. of gate lip} + r(1 - \cos\Theta)$$

$$= 2485.0 + 21.0\,(1 - \cos 15.0^\circ)$$

$$= 2485.75 \text{ ft. msl}$$

Items (5) to (10) were computed in the same manner as item (4) above.

HEAD DISCHARGE COMPUTATIONS FOR A DRUM GATE
IN RAISED POSITION

The discharge over the drum gate in the fully raised position as given in item (10), Cols. 4 and 5, was computed as described in the following steps:

1. Reservoir water surface elevations were assumed as given in Col. 1, Table 11.

2. The head on the gate was computed as the difference in elevation between the reservoir water surface in Col. 1 and the gate lip, and entered in Col. 2.

3. The ratios of the heads and the radius of the gate were computed and entered in Col. 3.

4. The discharge coefficient was determined from Plate 904 with the ratios of Col. 3 and with $\Theta = 34.9^\circ$, and entered in Col. 4

5. The discharge over the 64 foot drum gate was computed from Eq. 9-4 and entered in Col. 6.

(12)-(14) The discharges over the drum gate for other gate openings with positive gate lip angles were computed in the same manner as item (11) above.

(15)-(17) The discharges over the drum gate for negative gate lip angles were computed in the same general manner as for positive angles; except that the head was measured from the high point of the gate listed in Col. 5, Table I.

DISCHARGE RATING CURVE

(18) A discharge rating curve of the drum gate was plotted from the values of discharge, reservoir elevation, and gate elevation from Table II, and the results shown as seven curves on Plate 906 E. The extreme lower curve represents the discharge of the free crest with the gate completely down. The discharge values shown on Plate 906-E were for one gate only. A reasonable allowance for pier effect on the discharge was already present in the results.

DETERMINATION OF THE DISCHARGE RATING CURVE FOR BLACK CANYON DAM

Item

INITIAL DATA

(1) Profile of the spillway crest and 64 foot drum gate.

(2) Discharge rating curve for drum gates completely closed.

TABLE I

GATE ANGLE θ AND ELEVATION OF HIGH POINT OF GATE

Item	Gate lip elevation feet msl. Col. 1	Sine β Col. 2	β degrees Col. 3	Gate angle θ degrees Col. 4	Elevation of high point of gate feet msl. Col. 5
(3)	2478.2	-	-	-37.3	2482.5
(4)	2485.0	0.192	11.0	-15.0	2485.7
(5)	2487.0	0.305	17.8	- 8.3	2487.2
(6)	2489.0	0.419	24.8	- 1.3	2489.0
(7)	2491.0	0.535	32.2	+ 6.1	2491.0
(8)	2493.0	0.647	40.3	+14.2	2493.0
(9)	2495.0	0.760	49.5	+23.4	2495.0
(10)	2497.0	0.894	60.9	+34.9	2497.0

TABLE II

HEAD AND DISCHARGE COMPUTATIONS FOR A DRUM GATE IN RAISED POSITION

Item	Reservoir elevation feet msl. Col. 1	H feet Col. 2	H/r Col. 3	C_q Col. 4	$H^{1.5}$ Col. 5	Q cfs Col. 6
	Gate lip elevation 2497.0 ft.msl., θ = +34.9°					
(11)	2498.0	1	0.048	3.86	1	247
	2499.0	2	0.095	3.86	2.83	699
	2500.0	3	0.143	3.86	5.20	1,280
	Gate lip elevation 2495.0 ft.msl., θ = +23.4°					
(12)	2496.0	1	0.048	3.85	1	246
	2497.0	2	0.095	3.86	2.83	698
	2498.0	3	0.143	3.87	5.20	1,284
	2499.0	4	0.190	3.87	8.00	1,979
	2500.0	5	0.238	3.88	11.18	2,770
	Gate lip elevation 2493.0 ft.msl., θ = +14.2°					
(13)	2494.0	1	0.048	3.69	1	236
	2495.0	2	0.095	3.73	2.83	675
	2496.0	3	0.143	3.75	5.20	1,247
	2498.0	5	0.238	3.80	11.18	2,719
	2500.0	7	0.333	3.84	18.52	4,552
	Gate lip elevation 2491.0 ft.msl., θ = +6.1°					
(14)	2492.0	1	0.048	3.47	1	222
	2493.0	2	0.095	3.51	2.83	635
	2494.0	3	0.143	3.57	5.20	1,187
	2496.0	5	0.235	3.63	11.18	2,597
	2498.0	7	0.333	3.70	18.52	4,386
	2500.0	9	0.429	3.77	27.00	6,515

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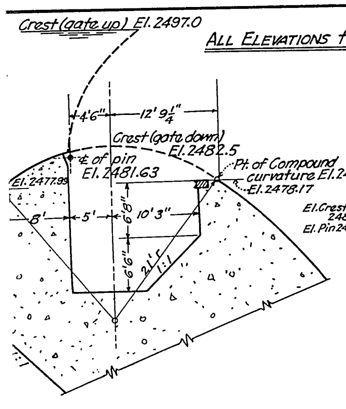

Fig. 1

SPILLWAY CREST DETAIL

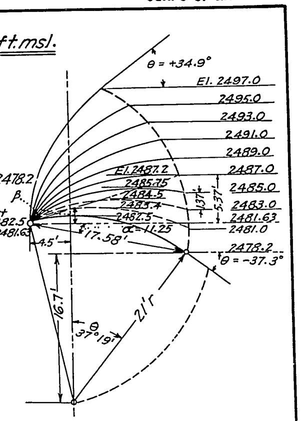

Fig. 2

RELATIONSHIP OF GATE ELEVATION TO ANGLE θ

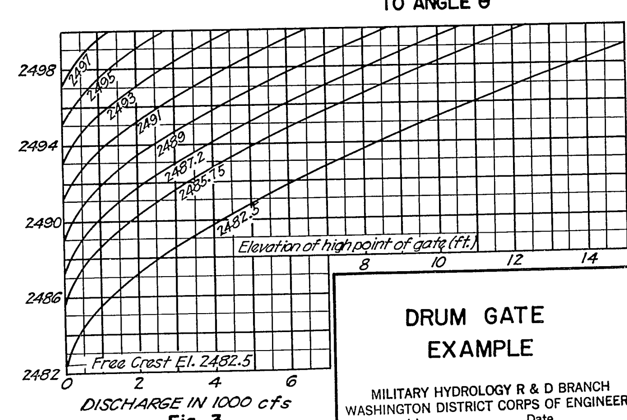

Fig. 3

DISCHARGE RATING CURVE FOR A DRUM GATE SPILLWAY

DRUM GATE EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ________
Drawn by ____________

TABLE II

HEAD AND DISCHARGE COMPUTATIONS FOR A DRUM GATE IN RAISED POSITION

Item	Reservoir elevation feet msl. Col. 1	H feet Col. 2	H/r Col. 3	C_q Col. 4	$H^{1.5}$ Col. 5	Q cfs Col. 6
	Gate high point elevation 2489.0 ft.msl., θ = -1.3°					
(15)	2490.0	1	0.048	3.21	1	205
	2491.0	2	0.095	3.28	2.83	594
	2492.0	3	0.143	3.34	5.20	1.110
	2494.0	5	0.238	3.45	11.18	2,470
	2496.0	7	0.333	3.545	18.52	4,200
	2498.0	9	0.429	3.63	27.00	6,270
	2500.0	11	0.524	3.695	36.48	8,630
	Gate high point elevation 2487.2 ft.msl., θ = -8.3°					
(16)	2488.0	0.8	0.038	3.02	0.72	138
	2489.0	1.8	0.086	3.10	2.42	479
	2490.0	2.8	0.133	3.17	4.68	950
	2492.0	4.8	0.229	3.31	10.52	2,230
	2494.0	6.8	0.324	3.43	17.73	3,890
	2496.0	8.8	0.419	3.51	26.10	5,860
	2498.0	10.8	0.515	3.58	35.49	8,130
	2500.0	12.8	0.610	3.635	45.79	10,650
	Gate high point elevation 2485.75 ft.msl., θ = -15.0°					
(17)	2487.0	1.25	0.060	3.00	1.40	268
	2488.0	2.25	0.107	3.07	3.38	663
	2489.0	3.25	0.155	3.15	5.86	1,180
	2491.0	5.25	0.250	3.275	12.03	2,520
	2493.0	7.25	0.345	3.375	19.52	4,220
	2495.0	9.25	0.440	3.465	28.13	6,240
	2497.0	11.25	0.536	3.54	37.73	8,550
	2499.0	13.25	0.631	3.595	48.23	11,100

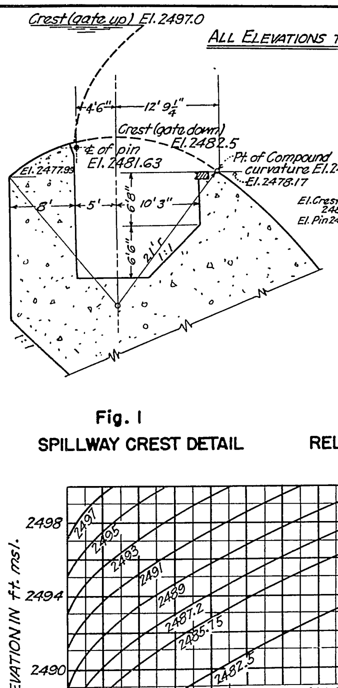

Fig. 1

SPILLWAY CREST DETAIL

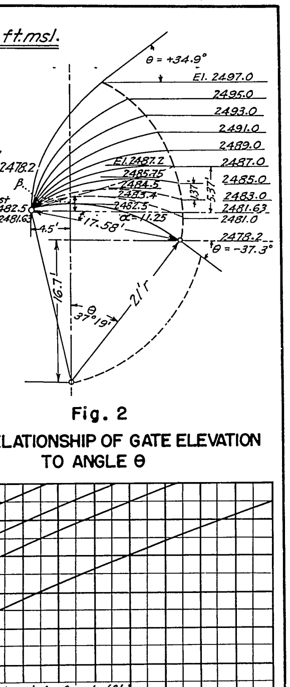

Fig. 2

RELATIONSHIP OF GATE ELEVATION TO ANGLE θ

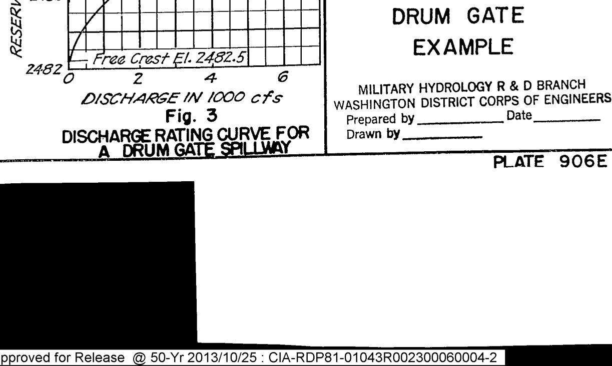

Fig. 3

DISCHARGE RATING CURVE FOR A DRUM GATE SPILLWAY

DRUM GATE EXAMPLE

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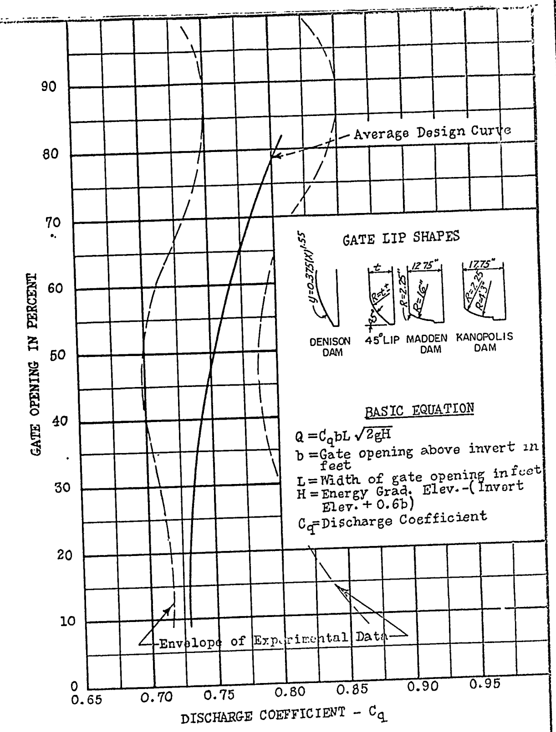
Drawn by __________



HIGH PRESSURE GATE DISCHARGE COEFFICIENT

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
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Drawn by ____________

PLATE 907

MHB-12

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DETERMINATION OF THE DISCHARGE RATING CURVE FOR A PARTIALLY OPENED HIGH PRESSURE OUTLET GATE

EXPLANATION OF COMPUTATIONS

Item

INITIAL DATA

(1)-(5) The basic data needed for the analysis of the problem are given on Plate 908 B.

(6) Manning's roughness coefficient was selected from Plate 501 to be 0.013.

HEAD LOSS COEFFICIENTS

(7) The total head loss in the conduit from the entrance to the partially opened gate was computed as the product of the sum of the velocity head coefficients, and the velocity head for each discharge.

1. Entrance loss coefficient (K_e). The entrance loss coefficient was determined from Plate 801 for a single gate passage on the design discharge curve.

$$K_e = 0.07$$

2. Friction head coefficient (K_f). The friction head coefficient was computed by Eq. 4-9 with C_f determined from the tabulated values in Par. 48 for n = 0.013.

$$R = A/P = 36/24 = 1.5 \text{ ft.}$$

$$R^{4/3} = 1.71$$

$$K_f = \frac{C_f L}{R^{4/3}} = \frac{(0.00492)\ (100)}{1.71}$$

$$K_f = 0.287$$

3. Gate recess coefficient (K_g). The gate recess loss coefficient was determined as described in Par. 140.

$$K_g = 0.10$$

4. The sum of the velocity head coefficients was

$$K = 0.46$$

DISCHARGE RATING CURVE

(8) The discharge rating curve of the high pressure outlet gate was determined as given in the following steps:

1. A gate opening of one foot was assumed and the percent of opening determined to be 1/6 or 16 percent. With a gate opening of 16 percent the coefficient of discharge was determined from Plate 907 to be 0.73.

2. Elevations of the energy gradient were assumed at a point immediately upstream from the partially opened gate and entered in Col. 1.

3. The vena contracts depth was determined as the product of the contraction coefficient and the gate opening and tabulated for each gate opening. The coefficient of contraction was taken as 0.6 for all ratios of head to gate opening.

4. The head on the gate was determined as the difference in elevation between the energy gradients of Col. 1 and the elevation of the top of the vena contracts and entered in Col. 2.

5. The discharge under the gate was computed by equation 9-5 and entered in Col. 3.

6. The average approach velocity and velocity head in the conduit for each of the discharge in Col. 3 were determined and entered in Col. 4 and Col. 5.

7. The total head loss was determined as the product of the velocity heads in Col. 5 and the sum of the velocity head coefficients of item (7), and entered in Col. 6.

8. The reservoir water surface elevations were computed as the sum of the head loss of Col. 6 and the elevation of the energy gradient of Col. 1, and entered in Col. 7.

9. The reservoir water surface elevation was plotted against the discharge and rating curve drawn.

Steps 1 through 9 were repeated for each gate opening and a family of curves plotted as shown.

DETERMINATION OF THE DISCHARGE RATING CURVE FOR A PARTIALLY OPENED HIGH PRESSURE OUTLET GATE

Item

INITIAL DATA

(1) Assume a 6 foot square horizontal concrete lined conduit 120 feet long.

(2) Bell mouth entrance without trash racks.

(3) Conduit outlet above tailwater.

(4) Vertical lift slide gate and guard gate 100 feet and 90 feet from conduit entrance respectively. (Guard gate fully open, regulating gate partially open).

(5) Gate seat elevation = 1000 feet msl.

(6) Manning's "n" = 0.013

HEAD LOSS COEFFICIENTS

(7) $H_L = K\, V^2/2g$ $\qquad K = K_e + K_f + K_g$

$$K_e = 0.07$$

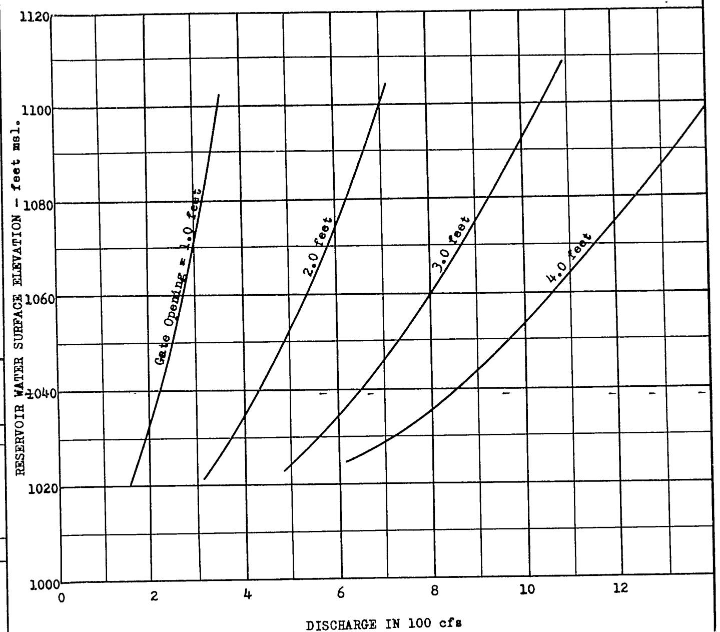
$$K_f = C_f \frac{L}{R^{4/3}} = 0.00492 \frac{100}{1.5^{4/3}} = 0.287$$

$$K_g = 0.10$$

$$K = 0.457$$

(8) DISCHARGE RATING CURVE

Energy Gradient El. feet msl. Col. 1	H feet Col. 2	Q = $C_q bL(2gH)^{0.5}$ cfs Col. 3	V feet/sec. Col. 4	$V^2/2g$ feet Col. 5	H_L = 0.46 $V^2/2g$ feet Col. 6	Reservoir W.S. Elev. feet msl. Col. 7
Gate opened 1.0 feet		C_q = 0.73	$C_c b$ = 0.6 feet		Q = 35.2 $(H)^{0.5}$	
1020.6	20.0	157	4.36	0.295	0.14	1020.74
1040.6	40.0	223	6.20	0.60	0.27	1040.87
1060.6	60.0	273	7.6	0.89	0.41	1061.01
1080.6	80.0	315	8.75	1.2	0.55	1081.15
1100.6	100.0	352	9.78	1.49	0.68	1101.28
Gate opened 2.0 feet		C_q = 0.74	$C_c b$ = 1.2 feet		Q = 71.3 $(H)^{0.5}$	
1021.2	20.0	318	8.85	1.22	0.56	1021.76
1041.2	40.0	450	12.5	2.43	1.12	1042.32
1061.2	60.0	552	15.4	3.68	1.69	1062.89
1081.2	80.0	637	17.7	4.86	2.23	1083.43
1101.2	100.0	713	19.8	6.2	2.81	1104.01
Gate opened 3.0 feet		C_q = 0.755	$C_c b$ = 1.8 feet		Q = 109.0 $(H)^{0.5}$	
1021.8	20.0	487	13.5	2.85	1.31	1023.11
1041.8	40.0	690	19.2	5.72	2.64	1044.44
1061.8	60.0	844	23.4	8.5	3.90	1065.70
1081.8	80.0	975	28.1	12.3	5.64	1087.44
1101.8	100.0	1090	30.3	14.2	6.55	1108.35
Gate opened 4.0 feet		C_q = 0.775	$C_c b$ = 2.4 feet		Q = 149.5 $(H)^{0.5}$	
1022.4	20.0	619	17.2	4.59	2.11	1024.51
1042.4	40.0	945	26.3	10.7	4.92	1047.32
1062.4	60.0	1160	32.3	16.2	7.42	1069.82
1082.4	80.0	1340	37.3	21.6	9.90	1092.30
1102.4	100.0	1495	41.6	26.9	12.40	1114.80



DISCHARGE RATING CURVES – PARTIALLY OPENED HIGH PRESSURE OUTLET GATE

PARTIALLY OPENED OUTLET GATE EXAMPLE

MILITARY HYDROLOGY R & D BRANCH
WASHINGTON DISTRICT CORPS OF ENGINEERS
Prepared by ____________ Date ________
Drawn by ____________

APPENDIX A

GLOSSARY OF LETTER SYMBOLS

The following letter symbols adopted for use in this bulletin are defined when they first appear in the text, and are arranged alphabetically in this appendix for reference. The letter symbols conform essentially with the American Standard Letter Symbols for Hydraulics (ASA-Z10. 2-1942), prepared by a committee of the American Standards Association and approved by the Association in 1942:

A = Area, cross-sectional

A_r = Area contraction ratio

A_t = Area of siphon throat

C = A general constant or coefficient
Chezy discharge coefficient

C_c = Coefficient of contraction

C_d = Coefficient of discharge at design head

C_q = Coefficient of discharge

C_s = Herschell's submerged discharge coefficient

D = Diameter

E = Energy
Crest rise of weir spillway

F = Total force
Fahrenheit

F_a = Total atmospheric pressure

F_h = Horizontal component of resultant pressure

F_n = Normal component of resultant pressure

F_r = Resultant of total pressure

F_v = Vertical component of resultant pressure

H = Total head

H_d = Total design head

H_o = Specific head or specific energy

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H_L = Total head loss

I = Moment of inertia (mass) or rectangular moment of inertia (area second moment) about any axis

$\overline{I}$ = Moment of inertia about the centroidal axis

K = Coefficient, velocity head

K_b = Bend velocity head coefficient

K_e = Entrance velocity head coefficient

K_f = Friction velocity head coefficient

K_g = Gate recess velocity head coefficient

K_{ge} = Gradual expansion velocity head coefficient

K_o = Outlet velocity head coefficient

K_r = Trash rack and stop log recess velocity head coefficient

K_s = Structure head loss coefficient

K_{sc} = Sudden contraction velocity head coefficient

K_{se} = Sudden expansion velocity head coefficient

K_v = Valve or obstruction velocity head coefficient

K_y = Branching pipe velocity head coefficient

L = Length

L_e = Equivalent length

M(y)= Pressure plus momentum factor as a function of depth

N = Submergence depth on a weir
Number of end contractions

N_f = Froude number

N_r = Reynolds number

O = Moment of area (statical or first moment) about any axis

P = Wetted perimeter

Q = Discharge

Q_c = Critical discharge

R = Hydraulic radius

S = Friction slope or energy grade line

S_c = Critical slope of a channel

S_o = Bottom slope

S_w = Water surface slope or hydraulic grade line

V = Mean velocity over cross section

V_c = Belanger's critical velocity

V_{cr} = Siphon-throat crest velocity

W = Total weight

Z = Elevation above datum plane

a = A constant
Acceleration

b = Breadth (width)
Height of gate opening
Bottom width

b_w = Water surface width of a channel

d = Diameter of orifice or valve outlet

f = Darcy-Weisbach coefficient

g = Gravitational acceleration

h = Head on weir or sluice gate
Piezometric head

h_a = Approach velocity head

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h_{at} = Atmospheric (absolute) pressure head

h_b = Head loss due to bends

h_d = Head drop on a submerged weir or bridge contraction

h_e = Head loss due to entrance conditions

h_f = Head loss due to surface resistance

h_g = Head loss due to gate recesses

h_{ge} = Head loss due to gradual expansions

h_L = Head loss

h_o = Head loss at outlet

$\bar{h}_p$ = Depth from water surface to centroid of area or opening

h_{po} = Pressure head at the throat crest of a siphon

h_r = Head loss due to trash racks and stop log recesses

h_{sc} = Head loss due to sudden contraction

h_{se} = Head loss due to sudden enlargement

h_t = Height of wind tide

h_v = Velocity head
Head loss due to valves or obstructions

h_{vc} = Velocity head at the throat crest of a siphon

h_w = Wave height

h_y = Head loss at a branching pipe

k = Radius of gyration about centroidal axis

k' = Conveyance of a cross section as a function of depth

m = Mass

m' = Critical depth factor as a function of depth

n = Manning's roughness coefficient

p = Pressure intensity per unit area

P_a = Atmospheric pressure

P_{abs} = Absolute pressure intensity

q = Rate of flow per unit width

r = General radius or radius of curvature

r_c = Syphon-throat crest radius

r_s = Siphon throat summit radius

s = Side slope (1 vert on s hor.)

t = Time
Thickness

x = Cartesian coordinate
Distance in direction of flow

y = Cartesian coordinate
Depth of open channel flow

y = Drop in water surface of an open channel

$\bar{y}$ = Distance from water surface to centroid of submerged surface along the prolongation line of the submerged surface

y_b = Depth at brink of overfall

y_c = Critical depth of flow

y_{cp} = Depth from water surface to center of pressure along prolongation line of submerged surface

y_o = Normal depth of flow

z = Cartesian coordinate
Weir crest height

$\Delta, \alpha, \beta, \theta, \phi$ = Angles

γ = Specific weight

ρ = Mass density = $\frac{\gamma}{g}$

APPENDIX B

GLOSSARY OF TERMS FOR HYDRAULICS

ACRE-FOOT.--Quantity of water that would cover 1 acre, 1 ft. deep. An acre-foot contains 43,560 cu. ft.

AFTER-BAY.--The tail-race of a water-power plant; a pond or reservoir at the outlet of the turbines.

ALTERNATE DEPTHS.--Two depths that have the same specific energy as shown by the specific energy curve for a constant discharge.

APRON.--A floor or lining of concrete, timber, etc., to protect a surface from erosion, such as the pavement below chutes, spillways, or at the toes of dams.

AQUEDUCT.--(1) A major conduit; (2) The entire transmission main for a municipal water supply which may consist of a succession of canals, pipes, or tunnels.

ARCHED DAM.--A curved dam, convex up stream, that depends on arch action for its stability, in which the load is transferred by the arch to the canyon walls or other abutments.

AREA CURVE.--(1) A graph of the cross-sectional area of a stream at a gaging station or other section; (2) a graph of the surface area of a reservoir plotted against water-surface elevations.

BAFFLE-PIERS.--Obstructions set in the path of high-velocity water to dissipate energy and thereby assist in preventing scour.

BARRAGE.--An obstruction placed in a watercourse, a dam.

BEAR-TRAP DAM.--An obstruction to flow built of hinged leaves, that are raised and held up by the pressure of water admitted to the inside.

BERNOULLI'S THEOREM.--A proposition (advanced by Daniel Bernoulli) that the energy head at any section in a flowing stream is equal to the energy head at any other downstream section plus the intervening losses.

BORE.--A wave of water having a nearly vertical front, such as the sudden release of a large volume of water from a reservoir. The bore is analogous to the hydraulic jump in that it represents the limiting condition of the surface curve wherein it tends to become perpendicular to the bed of the stream.

BOTTOM CONTRACTION.--The reduction in the area of overflowing water caused by the crest of a weir contracting the nappe.

BROAD-CRESTED WEIR.--An overflow structure on which the nappe is supported for an appreciable length in the direction of flow.

BUCKET.--A curved surface at the toe of an overflow dam designed to deflect the water horizontally; the transition curve between the overflow face and the apron of a dam.

CANAL.--An open conduit for the conveyance of water, usually excavated in natural ground.

CAPACITY CURVE.--A graph of the volume of a reservoir, as a function of the elevation.

CAVITATION.--A condition wherein a vacuum, to any degree, exists as a result of flowing water. Complete cavitation obtains when the pressure within the affected part is reduced to that of the vapor pressure of the water.

CHANNEL.--An elongated open depression in which water may flow.

CHUTE.--(1) A high-velocity conduit for conveying water to a lower level; (2) an inclined drop or fall.

COEFFICIENT OF DISCHARGE.--Ratio of observed to theoretical discharge. For a siphon this coefficient should be based on the area of the outlet.

COEFFICIENT OF ROUGHNESS.--A factor (in the Kutter, Manning, Bazin, and other formulas) expressing the character of a channel as affecting the friction slope of water flowing therein.

COFFER-DAM.--A barrier built in the water so as to form an enclosure from which the water is pumped to permit free access to the area within.

CONDUIT.--A general term for any channel intended for the conveyance of water, whether open or closed.

CONTRACTED WEIR.--A measuring notch with sides designed to produce a contraction in the area of the overflowing water.

CONTROL.--A section or a reach of a conduit where conditions exist that make the water level above it a fairly stable index of discharge. A control may be partial or complete. A complete control is independent of downstream conditions and is effective at all stages.

CREST.--The top of a dam, dike, spillway, or weir; frequently restricted to the overflow portion.

CRITICAL DEPTH.--The depth of water in which a given discharge flows in a given canal with a minimum content of specific energy.

CRITICAL FLOW.--A condition of flow in which the fluid is flowing in a canal at the critical depth.
See also, CRITICAL VELOCITY; SUB-CRITICAL FLOW; SUPER-CRITICAL FLOW.

CRITICAL VELOCITY.--Belanger's critical velocity is that condition in open channels for which the velocity head equals one-half the mean depth; that is, the mean velocity = $(gy_m)^{0.5}$.

DAM.--A barrier to confine or raise water for storage, diversion, or to create a hydraulic head.

DATUM.--Plane of reference for elevations.

DEBRIS.--Any material, including floating trash, suspended sediment, or bed load, moved by a flowing ssream.

DEBRIS DAM.--A barrier built across a stream channel to store debris.

DISCHARGE.--The quantity of water passing a section per unit of time.

DISCHARGE CURVE.--A rating curve showing the relation between stage or elevation and discharge.

DIVERSION DAM.--A barrier built for the purpose of diverting part or all the water from a stream into a different course.

DRAFT-TUBE.--An expending tube connecting the passages of a reaction waterwheel with the tailwater.

DRAINAGE AREA.--The area of a drainage basin or catchment area.

DROP.--A structure for dropping the water in a conduit to a lower level and dissipating its surplus energy. A drop may be vertical or inclined; the latter is called a "chute".

DRUM GATE.--A movable barrier in the form of a sector of a circle hinged at the apex. The arc face effects a water-seal with the edge of a recess into which the gate may be lowered. The gate is raised and held up by the pressure of water admitted to the recess from the headwater. It is lowered by closing the inlet port to the recess and draining the water from it.

EARTH DAM.--A barrier composed of earth, clay, sand, or sand and gravel, or a combination of earth and rock.

EDDY LOSS.--The energy lost (converted into heat) by swirls, eddies, and impact, as distinguished from friction loss.

END CONTRACTIONS.--The contraction in the area of overflowing water caused by the ends of a weir notch or bridge piers.

ENERGY.--The capacity to perform work: Kinetic energy is that due to motion; and potential energy is that due to position. In a stream the total energy at any section is represented by the sum of its potential and kinetic energies.
See also, ENERGY HEAD; ENERGY LINE; POTENTIAL ENERGY; KINETIC ENERGY.

ENERGY GRADIENT.--The slope of the energy line with reference to any plane.

ENERGY HEAD.--The elevation of the hydraulic grade line at any section plus the velocity head of the mean velocity of the water in that section. The energy head may be referred to any datum, or to an inclined plane, such as the bed of a conduit.

ENERGY LINE.--A line joining the elevations of the energy heads of a stream. The energy line is above the hydraulic grade line a distance equivalent to the velocity heads at all sections along the stream.

ENTRANCE HEAD.--The head required to cause flow into a conduit or other structure; it includes both entrance loss and velocity head.

ENTRANCE LOSS.--The head lost in eddies and friction at the inlet to a conduit or structure.

FISH LADDER.--A structure with pools and drops to facilitate the migration of fish around dams or other obstructions in streams.

FLASH-BOARD.--A plank, usually of timber and generally set on edge and held by supports on the crest of a dam or check structure, or in a spillway, to control the water level.
See also, NEEDLE; STOP-LOG.

FLUME.--An open conduit on a prepared grade, trestle, or bridge. A concrete-lined canal would still be a canal without the lining, but the lining supported independently would be a flume.

FOREBAY.--A reservoir or pond at the head of a penstock, pipe line, or pump station.

FREE-BOARD.--The distance between the normal operating level and the top of the sides of an open conduit or crest of a dam, to prevent overtopping the structure.

FREE FLOW.--A condition of flow through or over a structure not affected by submergence.

FRICTION HEAD.--The head or energy lost as the result of the disturbances set up by the contact between a moving stream of water and its containing conduit.

FRICTION SLOPE.--The friction head or loss per unit length of conduit. For most conditions of flow the friction slope coincides with the energy gradient, but where a distinction is made between energy losses due to bends, expansions, impacts, etc., a distinction must also be made between the friction slope and the energy gradient. Friction slope is equal to the bed or surface slope only for uniform flow in iniform channels.

GAGE HEIGHT.--The elevation of a water surface above or below a datum corresponding to the zero of the staff or other type of gage by which the height is indicated.

GAGING STATION.--A selected section in a stream channel equipped with a gage and facilities for measuring the flow of water.

GALLERY.-- A passageway in a dam.

GRAVITY DAM.--A dam depending solely on its weight to resist the water load.

HEAD.--The height of water above any point or plane of reference. Used also in various compounds, such as energy head, entrance head, friction head, static head, pressure head, lost head, etc.

HEAD-WATER.--The water upstream from a structure.

HYDRAULIC-FILL DAM.--A dam composed of earth, sand, gravel, etc., sluiced into place; generally the fines are washed toward the center for greater imperviousness.

HYDRAULIC FRICTION.--A flow-resisting force which is exerted on perimeter or contace surface between a stream and its containing channel. It usually includes the normal eddies and cross-currents attendant upon turbulent flow occasioned by the roughness characteristic of the boundary surface, moderate curvature, and normal channel variations.

HYDRAULIC GRADE LINE.--In a closed conduit a line joining the elevations to which water could stand in risers. In an open conduit, the hydraulic grade line is the water surface.

HYDRAULIC JUMP.--The sudden and usually turbulent passage of water from low stage below critical depth to high stage above critical depth during which the velocity passes from super-critical to sub-critical.

HYDRAULIC RADIUS.--The right cross-sectional area of a stream of water divided by the length of that part of its periphery in contact with its containing conduit; the ratio of area to wetted perimeter.

HYDROLOGY.--The science treating of the waters of the earth, their occurrence, distribution, movements, etc., often restricted to underground waters in distinction to hydrography as relating to surface water.

IMPACT.--The striking together of two masses. When particles or streams of water suffer inpact, energy losses result.

IMPACT LOSS.--The head lost as a result of the impact of particles of ~~water, included in~~ and ~~scarcely distinguishable~~ from eddy loss.

INLET.--The upstream end of any structure through which water may flow.

INTAKE.--The headworks of a conduit; the place of diversion.

INVERT.--The floor, bottom, or lowest part of the internal cross section of a conduit.

KINETIC ENERGY.--Energy due to motion. The kinetic energy of a given discharge is generally taken as proportional to the product of its weight per unit of time and the velocity head of its mean velocity. For a constant discharge, kinetic energy may be represented by a line at a distance above a flowing water surface proportional to the velocity head of its mean velocity. The elevation of such a line above any datum represents the total energy (potential plus kinetic) of the given discharge above that datum. Strictly, the kinetic energy of a given discharge is the integral of the kinetic energies of its particles.

LAMINAR FLOW.--That type of flow in which each particle moves in a direction parallel to every other particle, and in which the head loss is approximately proportional to the first power of the velocity. It is sometimes designated "stream-line flow" or "viscous flow".

LATERAL-FLOW SPILLWAY.--A spillway in which the initial and final flow are approximately at right angles to each other; a side-channel spillway.

MEAN DEPTH.--Cross-sectional area of a stream divided by its surface width.

MEAN VELOCITY.--(1) The velocity at a given section of a stream obtained by dividing the discharge of the stream by the cross-sectional area at that section; (2) Mean velocity may also apply to a reach of a stream by dividing the discharge by the average area of the reach.

MILITARY HYDROLOGY.--That branch of military engineering that includes all phases of hydrology and hydraulics relating to any aspect of runoff, streamflow and groundwater. It has an important direct effect on military planning and operations, and includes artificial floods caused by the destruction or manipulation of water control structures.

MULTIPLE ARCH DAM.--A barrier consisting of a series of arches supported by buttresses or piers. The load is transferred by the several arches to the foundation through the buttresses.

NAPPE.--A sheet or curtain of water overflowing a weir, dam, etc. The nappe has an upper and a lower surface.

NEEDLE.--A timber set on end to close an opening for the control of water; it may be either vertical or inclined.

NON-UNIFORM FLOW.--A flow the velocity of which is undergoing a positive or negative acceleration. If the flow is constant it is referred to as "steady non-uniform flow".

NORMAL DEPTH.--The depth of water in an open conduit that corresponds to uniform velocity for the given flow. It is a hypothetical depth under conditions of steady non-uniform flow; the depth for which the surface and bed are parallel.

OGEE.--The reversed curve of the face of an overflow dam.

ORIFICE.--A hole or opening, usually in a plate, wall, or partition, through which water flows, generally for the purpose of control or measurement.

OVER-FALL.--The part of a dam or weir over which the water pours.

PARABOLIC WEIR.--A measuring weir whose notch is bounded on the sides by parabolas such that the flow is proportional to the head.

PARSHALL MEASURING FLUME.--A device or control flume to measure the flow of water in open conduits. It consists essentially of a contracting length, a throat, an expanding length. At the throat is a sill over which the water is intended to flow at Belanger's critical depth. The upper head is measured a definite distance upstream and the lower head a definite distance downstream from the sill. The lower head need not be observed except where the sill is submerged more than about 67 percent.

PENSTOCK.--A closed conduit for supplying water under pressure to a waterwheel or turbine.

PIEZOMETER.--An instrument for measuring pressure head, usually consisting of a small pipe tapped into the side of a closed or open conduit and flush with the inside, connected with a pressure gage, mercury, water column, or other device for indicating pressure head.

PITOT TUBE.--A device for observing the velocity head of flowing water, consisting essentially of an orifice held to point upstream in flowing water and connected with a tube by which the rise of water in the tube above the water surface may be observed. It may be constructed with an upstream and a downstream orifice and two water columns, the difference of water levels being an index of the velocity head.

POTENTIAL ENERGY.--Energy due to position. The potential energy of a given volume of immobile water with reference to any datum, is proportional to the product of its weight and the elevation of the center of gravity above that datum. The potential energy per unit of time of a given discharge at any instant with reference to any datum, is proportional to the product of its weight per unit of time and the elevation of its hydraulic grade line above that datum, at that instant.

PRESSURE.--Total load or force acting upon a surface; also appropriately used to indicate intensity of pressure or force per unit area.

PRESSURE HEAD.--The head on any point in a conduit represented by the height of the hydraulic grade line above that point.

RADIAL GATE.--A pivoted gate whose face is usually a circular arc with center of curvature at the pivot; a tainter gate.

RATING CURVE.--A graphic representation of a rating; a calibration.

REACH.--A comparatively short length of a stream or channel.

RIGHT BANK OF STREAM.--The right-hand bank when the observer is looking downstream.

ROCK-FILL DAM.--A dam composed of loose rock usually dumped in place; often with the upstream part constructed of hand-placed, or derrick-placed, rock and faced with rolled earth or with an impervious surface of concrete, timber, or steel.

ROLLER GATE.--A hollow cylindrical gate with spur gears at each end meshing with an inclined rack anchored to a recess in the end pier or wall. It is raised or lowered by being rolled on the rack. It may close at a greater depth than its diameter by means of a shield or apron attached to the cylinder.

SCOURING SLUICE.--An opening in a dam controlled by a gate through which the accumulated silt, sand, and gravel may be ejected.

SECTOR GATE.--A roller type of gate in which the roller is a sector of a circle instead of a complete cylinder.

SEQUENT DEPTHS.--The depths before and after the hydraulic jump. The depth after the hydraulic jump is less than the alternate depth because of the loss of energy in the jump.

SHARP-CRESTED WEIR.--A measuring weir with its crest at the upstream edge or corner of a relatively thin plate, generally of metal.

SHOOTING FLOW.

See SUPER-CRITICAL FLOW.

SIDE-CHANNEL SPILLWAY.

See LATERAL-FLOW SPILLWAY.

SIDE SLOPES.--The slppe of the sides of a canal, dam, or embankment; custom has sanctioned the naming of the horizontal distance first as 1.5 to 1.

SIPHON.--A closed conduit, a part of which rises above the hydraulic grade line.

SLUICE.--An opening in a structure for passing water at high velocities for wastage.

SPECIFIC ENERGY.--The energy of a stream referred to its bed; namely, depth plus velocity head of mean velocity.

SPILLWAY.--A passage for spilling surplus water; a wasteway.

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STAGE.--The elevation of a water surface above its minimum; also above or below an established "low-water" plane; hence, above or below any datum of reference; gate height.

STATIC HEAD.--The total head without deduction for velocity head or losses; for example, the difference in elevation of headwater and tailwater of a power plant.

STEADY FLOW.--A constant flow; that is, the same volume in equal units of time.

STILLING BASIN.--A transition structure wherein the partial kinetic energy of supercritical flow is converted into turbulent energy and heat by the hydraulic jump, and the flow is discharged downstream in a tranquil or subcritical state.

STILLING WELL.--A pipe, chamber, or compartment with closed sides and bottom except for a comparatively small inlet or inlets communicating with a main body of water. Its purpose is to dampen waves or surges while permitting the water-level within the well to rise and fall with the major fluctuations of the main body.

STOP-LOG.--A log, plank, cut timber, steel or concrete beam fitting into end guides between walls or piers to close an opening to the passage of water; usually handled one at a time.

STREAMING FLOW.

See SUB-CRITICAL FLOW.

SUB-CRITAL FLOW.--Turbulent flow with a mean velocity less than Belanger's critical velocity.

SUBMERGENCE.--The ratio of the tailwater elevation to the headwater elevation, when both are higher than the crest, the overflow crest of the structure being the datum of reference.

SUPER-VRITICAL FLOW.--Turbulent flow with a mean velocity equal to or greater than Belanger's critical velocity.

SUPPRESSED WEIR.--A measuring weir notch whose sides are flush with the channel, thus eliminating (suppressing) end contractions of the overflowing water. A weir may be suppressed on one end, two ends, bottom, or any combination of them.

SURFACE CURVE.--(1) The longitudinal profile assumed by the surface of a stream of water flowing in an open conduit; (2) The hydraulic grade line.

SURFACE SLOPE.--The inclination of the water surface expressed as change of elevation per unit of slope length.

TAIL-RACE.--A channel conducting water away from a water wheel; an after-bay.

TAILWATER.--The water just downstream from a structure.

TAINTER GATE.

See RADIAL GATE.

THALWEG.--A line following the lowert points of a channel.

TRANSITION.--A short conduit uniting two others having different hydraulic elements.

TRAPEZOIDAL WEIR.--A contracted measuring weir with a trapezoidal notch.

TRASH RACK --A grid or screen across a stream designed to catch floating debris.

TRIANGULAR WEIR.--A contracted measuring weir notch with sides that form an angle with apex downward; the crest is the apex of the angle; a V-notch weir.

TURBULENCE.--A state of flow wherein the water is agitated by cross-currents and eddies; opposed to a condition of flow that is quiet or quiescent.

UNIFORM FLOW.--A constant flow or discharge, the mean velocity of which is also constant.

V-NOTCH WEIR.

See TRIANGULAR WEIR.

VELOCITY HEAD.--The distance a body must fall zreely under the force of gravity to acquire the velocity it possesses.
See also, KINETIC ENERGY.

VELOCITY OF APPROACH.--The mean velocity in the conduit immediately upstream from a structure.

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VENA CONTRACTA.--The most contracted sectional area of a stream, jet, or nappe beyond the plane of the orifice, or notch, through which it issues.

VENTURI METER.--A proprietary measuring device, consisting essentially of a Venturi tube and a special form of flow-registering device.

VENTURI TUBE.--A closed conduit which is gradually contracted to a throat causing a reduction of pressure head by which the velocity through the throat may be determined. The contraction is generally followed, but not necessarily so, by gradual enlargement to original size. Piezometers connected to the pipe above the contracting section and at the throat indicate the drop in the pressure head which is an index of flow.

VERTICAL-VELOCITY CURVE.--A graph of the relation between depth and velocity along a vertical line in a stream, as determined by a set of observations.

WATER-HAMMER.--The phenomena of oscillations in the pressure of water in a closed conduit, resulting from checking the flow.

WEIR.--A dam across a stream for diverting or for measuring the flow.

WETTED PERIMETER.--The length of the wetted contace between a stream of water and its containing conduit, measured along a plane at right angles to the direction of flow; that part of the periphery of the cross-sectional area of a stream in contact with its container.

WICKET-DAM.--A movable barrier made of wickets, or shutters, revolving about a central axis.

APPENDIX C

ASSOCIATED MILITARY HYDROLOGY PUBLICATIONS

1. MHB 1: Applications of Hydrology in Military Planning and Operations
2. MHB 2: River Characteristics and Flow Analyses for Military Purposes
3. MHB 3: Stream-Gaging Methods and Equipment for Military Purposes
4. MHB 4: Transmission of Hydrologic Data for Military Purposes
5. MHB 5: Card-Indexing and Filing of Information Pertinent to Military Hydrology
6. MHB 6: Directory to European Sources of Information on Military Hydrology
7. MHB 7: Glossary of Terms Pertinent to Military Hydrology
8. MHB 8: Selected References on Military Hydrology
9. MHB 9: Flow Through a Breached Dam
10. MHB 10: Artificial Flood Waves
11. MHB 11: Regulation of Stream Flow for Military Purposes
12. MHB 12: Handbook of Hydraulics